PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 66 | OCTOBER, 1940 | NO. 8, PART 1

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

THE GRAND CENTRAL TERMINAL IN PERSPECTIVE

By William J. Wilgus,[1] Hon. M. Am. Soc. C. E.

Synopsis

This paper is intended more as an exposition of the inception, development, and quickening of a complex organism of magnitude, dating from 1831, than as a detailed description of the manner in which each of its many parts has been designed and constructed. Within this framework appears an outline of the results that have been achieved in the interest of the owner, and of the public to whom they spell a successful adventure in civic planning—all made possible by the utilization of overhead air rights incident to the change of motive power from steam to electricity. Included also is an intimation of problems ahead, to be faced and solved.

To the engineer who reads the paper will perhaps come the reminder that men of his profession, endowed with vision, may evoke as well as direct "the great sources of power in Nature for the use and convenience of man," without necessarily the promise of self-enrichment customarily demanded by the business man of genius. To him, moreover, will come the thought that it is not only the railroad station he traverses, visible to the eye, that is affording him service and is entitled to his consideration, but also the vast unseen plant of which the station itself is the outward sign.

Introduction

Purpose to Be Served.—The completion of engineering works of magnitude is usually marked by their description in the annals of the Society or in the technical press. The calling of the engineer is thereby enriched, and its claim to inclusion among the learned professions is increasingly strengthened. In one respect, however, it would seem that this laudable practice has fallen short of perfection. Seldom, if ever, do after years bring forth a subsequent statement of the extent to which the work in question may have measured up to

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by February 15, 1941.

[1] Weathersfield, Ascutney, P. O., Vt.

expectations in whole or part. Members of the profession and the public at large too frequently are not apprized of its merits and faults, of which adequate knowledge is to be gained only through long-time experience. To be fairly judged and truthfully recorded on the pages of history, the work should be seen in perspective. It is to the interest of the owner, as well as of the engineering profession, that a deserved reputation for having well served a public purpose shall be made known, apart from the gaining of a monetary reward which also has proved in the end to have crowned the work with success.

It is in this spirit that the writer is venturing to lay before the profession the evolution of the Grand Central Terminal and its approaches in New York, N. Y., and vicinity, not alone because of the service that they have rendered to their owners and the travelers who use them, but also because of their contribution to enlightened civic planning.

Early Background from 1831 to 1869

Beginnings of the New York and Harlem Railroad.—The marvelous growth of the City of New York, confined within the limits of the island of Manhattan until 1874, and expanded to include the other four boroughs (Brooklyn, Queens, Bronx, and Richmond) after 1898, is matched by the striking advances that have been made in its means of trunk-line transportation since they had their start as a horsecar railroad more than a century ago.

In 1831, shortly after steam railroading devoted to public service first came to life in the United States, the New York and Harlem Railroad was incorporated; and its initial stretch of a little less than a mile was put into use with horses as the motive power between Prince and 14th streets in the following year. At that time the city's population was not much in excess of 200,000, and its projected street system north of 34th Street was as yet unopened and ungraded. Six and a half miles away was the isolated little village of Harlem, and between it and the city was the hamlet of Yorkville. The rugged mid-island terrain that faced the infant enterprise was indeed formidable for the times. Five years were required for the road to reach Harlem, in 1837, and another five to bridge the Harlem River and arrive at the point north of Williams Bridge, in 1842, where later the New York and New Haven Railroad, in 1849, was to make its connection at Woodlawn Junction, N. Y. Meantime, in 1839, the road had been extended southerly from Prince Street for a little more than a mile to the City Hall, and still farther in 1851 to the Astor House. Twenty years had elapsed since the work was started. In another year (1852) the road had completed its way to a junction with the Boston and Albany Railroad at Chatham, 130.75 miles from its southern terminus in New York.

In the initial distance of nearly 16 miles, double-tracked by this time as far as Woodlawn Junction, the larger portion of the line south of the Harlem River had its place in city streets, either from the beginning, as for example in the Bowery, or later in Fourth Avenue and its continuation (subsequently to be known as Park Avenue) north of 32d Street after the company's independent 100-ft right of way by agreement with the city in 1837 had been incorporated within that thoroughfare, and then widened north of 34th Street to 140 ft.

The motive power of the road was limited by law to the use of horses south of 14th Street. North of that point, as a result of public pressure, the steam locomotive was constantly on the retreat, first to an engine terminal at 32d Street in 1846 and then to a location just north of 42d Street in 1859, from whence, a half a century later, it was to take its flight to far-off South Croton-on-Hudson (Harmon), N. Y., and North White Plains, N. Y. So for nearly forty years, from 1832 to 1871, railroad cars in Manhattan were individually drawn by horses for increasing distances as the use of the steam locomotive became more and more obnoxious to a population that, by 1870, had grown to approximately a million people.

Entry of the New York and New Haven Railroad.—The New York and New Haven Railroad, a link in a series of railroads originating at Boston and other points in New England, secured trackage rights over the Harlem road south of Woodlawn Junction on March 17, 1848, and in the next year for the first time operated its trains in this manner for about 15 miles to Canal Street, where its terminus, adjoining a station of the Harlem road, remained until 1857. In that year the two roads established separately operated passenger stations side by side, with twelve tracks in all, on a block on the west side of Fourth Avenue between 26th and 27th streets, reaching back to Madison Avenue immediately north of Madison Square; no buildings had as yet been erected or streets opened above 42d Street. There the principal railroad terminus in mid-Manhattan remained for fourteen years until the Grand Central Depot took its place, and the use of horses for hauling trunk-line cars on the Harlem road south of 42d Street was thenceforth abolished. It was the focus of the rapidly growing trade that had been fostered and developed by the widening network of railways in New England, shortly to be expanded by extensions to Montreal and Quebec, Canada.

Arrival of the Hudson River Railroad.—It now remains to sketch the origin and development of Manhattan's third railroad entrance skirting its western shore (see Fig. 1). Incorporated in 1846 under the name of the Hudson River Railroad, and completed from Canal Street to Spuyten Duyvil, N. Y., in 1847, it was opened for its entire length, 144 miles, to East Albany, N. Y., in 1851, two years after the two mid-Manhattan roads had put the city in rail communication with the New England states. Beginning at Chambers Street, quite a distance west of the City Hall terminus of the New York and Harlem Railroad, it took its course at grade through Hudson, Canal, and West streets; then through 10th Avenue and private property to a station at 30th Street, to which point it was operated first by horses and eventually by means of a "dumb engine" preceded by a man on horseback bearing a red flag; and thence in 11th Avenue, ultimately to be known popularly as "Death Avenue," and beyond, the trains were hauled by steam locomotives of the usual type.

Shortly after the completion of the Hudson River Railroad under the direction of John B. Jervis,[2] Hon. M. Am. Soc. C. E., the series of lines that joined Albany, N. Y., with Buffalo, N. Y., were consolidated, in 1853, under the name of the New York Central Railroad, and by means of a ferry across

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 109.

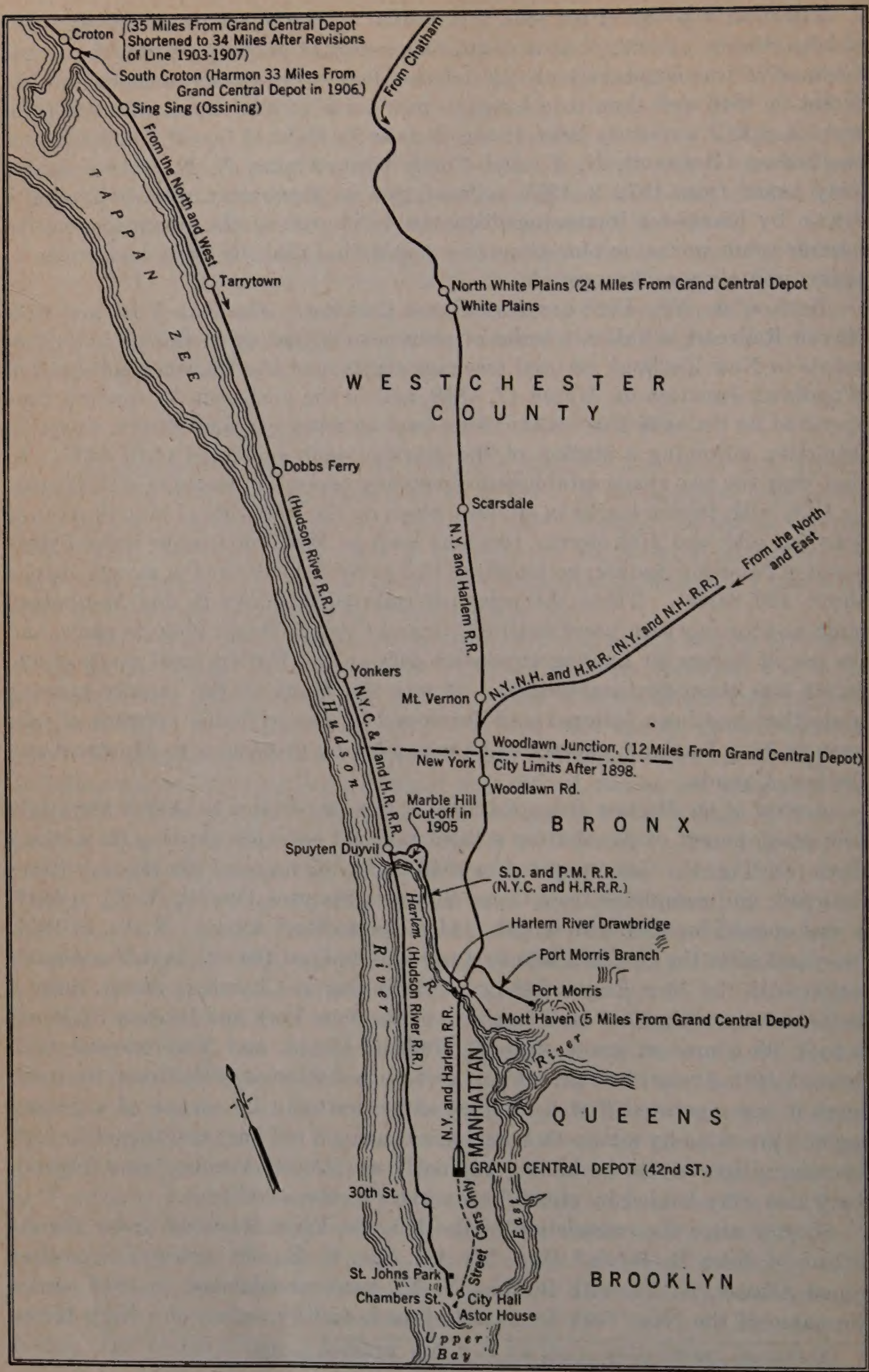

FIG. 1.—RAILROAD ENTRANCES TO MANHATTAN, 1871–1874

the Hudson River at Albany were connected with the Hudson River Railroad to create a rail route from Chambers Street, in New York City, to Buffalo.

Birth of the New York Central and Hudson River Railroad.—The time now was ripe for the entry into the situation of a man of force and vision who could and would seize the opportunity of welding together and improving these separately controlled arteries centering in Manhattan from the north. Cornelius Vanderbilt proved to be the man. Between 1863 and 1867 he had acquired control of the New York and Harlem, the Hudson River, and the New York Central railroads, and on November 1, 1869—the year in which the nation was first spanned by rail from sea to sea—had brought about the consolidation of the latter two under the name of the New York Central and Hudson River Railroad. Thereupon he began the construction of a link between that road at Spuyten Duyvil on the Hudson River and his controlled New York and Harlem Railroad at Mott Haven, N. Y. In bringing the traffic of all three systems on the north into a funnel to be used by them in common for 5 miles of the Harlem's line as far as 42d Street, he also planned the building of a terminus, to be known as the Grand Central Depot. South of the depot the movement of their cars was to cease. With this went the concept that the then existing two-track surface line between the Harlem River and the new terminal should be four-tracked and depressed beneath the surface of intersecting streets in open cuts south of 56th Street and north of 96th Street, and in a tunnel under Fourth (Park) Avenue for the intervening distance.

Concept of a Grand Central Depot and Improved Approaches.—In 1869, therefore, the original Grand Central Depot and its improved approaches were visualized by Commodore Vanderbilt. He lived to see them partly completed (he died in 1877)—a fitting means of access to the nation's metropolis by three railroad systems which then reached far in all directions to the north, west, and northeast, and promised to expand farther with the coming years. This is the background, a knowledge of which has seemed necessary for a proper understanding of the course of events that have had their culmination in the Grand Central Terminal of the present day.

Grand Central Depot from 1869 to 1899

Planning and Building the Depot and Yard.—The decision having been made in 1869 that the new terminus should be located on the site of the then existing steam locomotive facilities between 42d and 45th streets (see Fig. 2), the purchase of the needed additional lands between Madison and Lexington avenues as far north as 48th Street was undertaken. By October 7, 1871, the work was well advanced, and the announcement was made that the trains of the Harlem road, one of which had departed from the depot on that day, would begin regularly to arrive and depart from it two days later, to be followed a week thereafter by those of the New Haven road, and still a week later by those of the New York Central. Apparently the link along the east bank of the Harlem River connecting the tracks of the former Hudson River Railroad at Spuyten Duyvil with those of the New York and Harlem at Mott Haven was then sufficiently ready for use, although according to some accounts it was not until the following year that it was fully completed.

In completing this plan and that which was soon to follow, a four-track open cut between retaining walls was created in Park Avenue from 56th Street to 49th Street, south of which the yard, embracing some 10 miles of track, fanned out on the surface for storing cars as far as Madison Avenue north of

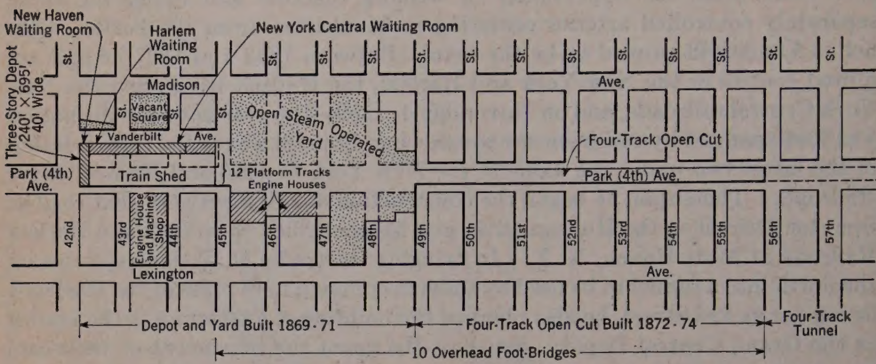

FIG. 2.—MAP OF DEPOT FACILITIES, 1871–1874

45th Street and for serving the depot with its frontage of 240 ft on the north side of 42d Street and 695 ft on the east side of Vanderbilt Avenue. The work, spreading over two years, was under the direction of Commodore Vanderbilt and his son, William H. Vanderbilt, and was designed and its construction supervised by Isaac C. Buckhout,[3] M. Am. Soc. C. E.

Defects in Design.—The new depot when opened was hailed by the press of the day as deserving the thanks of the traveling community for such a liberal provision for their wants, and the arched roof of the train shed was praised as magnificent in its stately dimensions and its sincerity and dignity of design (see Fig. 3). In responsible quarters, however, it was criticized severely for its want of unity in its different parts, its unattractive architectural design, and its "unfortunate" exterior color treatment. In one important particular it was spoken of as "a great blunder." The yard tracks on the surface acted as a veritable "Chinese wall" to separate the city into two parts for fourteen blocks —nearly three quarters of a mile—between 42d Street and 56th Street and forced the discontinuance of a leading north and south thoroughfare, then known as Fourth Avenue, between 42d and 49th streets. In time, too, the rising 1¼% gradient south of 56th Street, and the absence of independent switching leads alongside of the four-track main entrance, were to limit gravely the capacity of the terminal, which in a few years was to be called upon to handle two to three times the 164 trains per day that greeted the opening in 1871. In effect, the entrance to the depot yard was throttled down from four main tracks to two, south of 56th Street.

Fourth Avenue Improvement.—The Grand Central Depot was no sooner completed than plans were laid for the four-tracking of its approach as far north as the Harlem River. This was known as the Fourth Avenue Improvement and involved the building of an open cut between retaining walls from 49th Street

[3] For memoir, see *Transactions*, Am. Soc. C. E., Vol. I (1872), p. 171.

to 56th Street (see Fig. 2); thence a tunnel to 96th Street; thence an open cut and a stone-arch viaduct to 115th Street; and thence an open cut to 133d Street, a monumental work, the cost of which was divided between the railroad company and the city. Its execution was entrusted to four commissioners, Alfred W.

FIG. 3.—VIEW OF TRAIN SHED, LOOKING SOUTH, 1871

Craven,[4] Past-President, Am. Soc. C. E., Allan Campbell,[5] Hon. M. Am. Soc. C. E., and Edward H. Tracy,[6] M. Am. Soc. C. E., with Mr. Buckhout acting also as the engineer in charge. These four, and others on the work, such as Fayette S. Curtis,[7] Past-President, Am. Soc. C. E., and S. LeF. Deyo,[8] M. Am. Soc. C. E., in later years became prominent in the Society's affairs.

The construction of this adjunct to the usefulness and efficiency of the Grand Central Depot consumed about two years, from 1872 to 1874, during which, in 1872, the New York and New Haven Railroad became a part of the New York, New Haven and Hartford Railroad system; and, in 1873, the New York and Harlem Railroad was leased to the New York Central and Hudson River Railroad Company. Its outcome in a few years was to draw upon the owning company the opprobrium of the public at large and of passengers who were forced to travel through the covered section. The products of locomotive combustion in the two single-track side tunnels were imperfectly discharged by natural ventilation into the double-track center tunnel, from whence the smoke, gas, and cinders from all three tunnels were shot upward through longitudinal openings between the streets into the faces of passers-by and the occupants of the abutting property. The obscuration of signals in the tunnel and the dis-

[4] For memoir, see *Proceedings*, Am. Soc. C. E., Vol. VI (1880), p. 24.

[5] For memoir, *loc. cit.*, Vol. XX, October, 1894, p. 179.

[6] For memoir, see *Transactions*, Am. Soc. C. E., Vol. I, p. 377.

[7] For memoir, *loc. cit.*, Vol. 94 (1930), p. 1450.

[8] For memoir, *loc. cit.*, Vol. LXXXVI (1923), p. 1646.

tressing conditions to which the passengers were subjected in hot weather, together with injuries inflicted on the surroundings, all had their part in the initiation of the plan for a change of motive power from steam to electricity in 1899.

Yard Improvements, Including the Annex.—With the passing of years other improvements were found to be necessary in order properly to meet a rapidly growing business, of which the first, in 1885–1886 (see Fig. 4), was the installation of improved interlocking switches and signals in an enlarged and rearranged

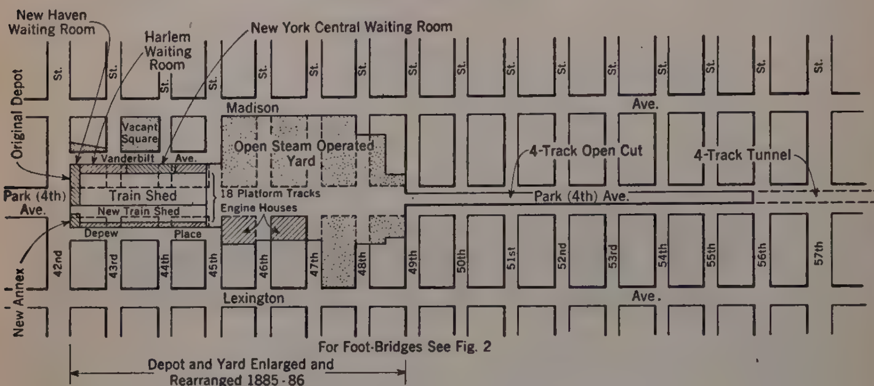

Fig. 4.—Map of Depot Facilities, 1885–1886

yard. With this went the erection of a seven-track Annex, 100 ft wide and 650 ft long, on the easterly side of the depot for the benefit of incoming trains. The depot by this time contained twenty tracks of which eighteen were alongside of platforms. By an agreement dated December 31, 1885, horsecars were to be excluded from the depot confines, and a yard was to be provided at Mott Haven, five miles away, for the repair, cleaning, and storage of the New York Central's locomotives and through trains.

Harlem Depression.—The next change of magnitude was the depression and four-tracking of the Harlem road for 5 miles between a point just north of 138th Street and Woodlawn Road. This entailed the installation of automatic block signals, the elimination of grade crossings at twenty-two streets, and the building of five overhead passenger stations. The work lasted from 1888 to 1890 and was in charge of the chief engineer of the railroad, Col. Walter Katté,[9] M. Am. Soc. C. E. In 1891 improved signals were installed in the Park Avenue tunnel.

Park Avenue Viaduct and Harlem River Drawbridge.—The roadway, thus widened for four tracks at great expense, was not to be permitted to "rest in peace," however. Bending to popular clamor, the railroad officials, in the years 1893 and 1894, joined with the city in raising the tracks out of the depression in Park Avenue, between 106th Street and the Harlem River, and placing them on a four-track steel viaduct longer than 6,400 ft, north of 110th Street. Beyond the viaduct the railroad, at its own expense, substituted a

[9] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXXI, December, 1917, p. 1727.

high-level, four-track drawbridge, 398 ft long, with two approach spans on the north, for the old low-level bridge that had been a serious cause of train delays. The viaduct work, practically completed in March, 1895, was prosecuted under a "Board for the Park Avenue Improvement above 106th Street," for which Colonel Katté was the superintending engineer. In this he was aided by George H. Thomson,[10] M. Am. Soc. C. E., engineer of bridges, as he was in the case of the drawbridge and its approaches, completed in May, 1896, in which he acted as chief engineer for the railroad.

Additional Stories and "Face Lifting."—The next move for improvement in the terminal situation was the enlargement of the depot building, by adding to it three stories for office use, and by altering its architectural form and external appearance through the substitution of an artificial stone-work, stucco finish for the original frankly ugly natural brick. This change, completed in 1898, was made under the direction of Colonel Katté, chief engineer, and Bradford L. Gilbert, architect.

Union Waiting Room, Track Changes, and Baggage Subway.—Then came the final alteration in the original concept when, in 1900 (see Fig. 5), the layout in the train shed was rearranged so as to provide for eleven platforms, served by

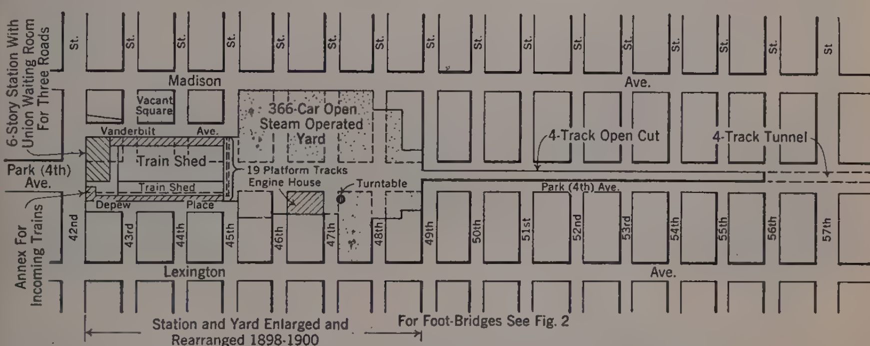

FIG. 5.—MAP OF DEPOT FACILITIES, 1898–1900

nineteen tracks under cover, and a baggage subway at their north end, coupled with the slight shortening of the stub tracks at their south end in order that a 90-ft by 180-ft single waiting room, 36 ft high, and a 30-ft concourse might be provided between them and 42d Street, in place of the individual waiting rooms that had been used separately by the three companies during the previous twenty-nine years. Extensive changes, too, were made in the yard, including improvements in the signals and interlocking. The writer, as chief engineer, and Samuel Huckle, Jr., as architect, were responsible for the planning and direction of the station changes which, by the way, had the endorsement of Mr. Curtis, then chief engineer of the New Haven company. At last the depot (Fig. 6) with its yard (Fig. 7) was to be used in common by the owner and its two tenants as a union station in the full meaning of the term. Its name had become the Grand Central Station and it looked as if, with its approaches from the north, it had reached a stage of perfection, barring the dreadful smoke con-

[10] For memoir, *Transactions*, Am. Soc. C. E., Vol. LXXI, March, 1911, p. 438.

ditions in the yard and Park Avenue tunnel, the severance of the city's street system in a region of rapidly growing importance, and the lack of switching leads at the throat of the yard. Certainly there was nowhere in the United

Fig. 6.—View of the Grand Central Station, Looking Northeast, 1898–1900

Fig. 7.—Yard Layout in 1906, Looking South

States, if in the world, a more noteworthy railroad terminal, with its enormously costly 12-mile, four-track entrance without grade crossings from New England and Canada, connecting 5 miles out with a double-track line bearing traffic

from the north and west as far as the St. Lawrence River and the Great Lakes and beyond to sub-arctic regions and the Pacific Coast.

Grand Central Terminal Transformation—Formative Period from 1899 to 1907

Initial Move for Electrification.—In 1898, the writer, as engineer of maintenance of way, New York Central Railroad system, was required to rehabilitate the entire system for heavy motive power and the other requirements of modern times, and in the spring of the following year it fell to him, in his new position of chief engineer, to devise a means for improving conditions in and approaching the Grand Central Station. The dangers and discomforts in the Park Avenue tunnel were calling aloud for abatement; the yard with its fly-switch operations and upward of 500 trains a day, nearly three times the number that were handled in the 1870's, had become inadequate; the public was pressing for the restoration of cross streets and the abolition of the smoke nuisance at the yard; and the needs of the three railroad companies for more office space, despite the recently completed enlargement of the depot building, were becoming more and more apparent. Then, too, the Board of Rapid Transit Commissioners were intimating that they might by law secure the right to route their subway beneath the terminal, to the exclusion of such underground usage as the railroad might find to be necessary in the future.

The writer's modest experience with electrical problems, gained in the mid-west, was fanned anew into flame by a visit from Frank J. Sprague,[11] M. Am. Soc. C. E., who called on the writer to propose the electrification of the company's branch to Yonkers, N. Y. This conference brought the writer to the conclusion that suburban trains at least might be operated southerly through the side tunnels to 56th Street and thence by means of additional tracks in a widened open cut in Park Avenue to a loop station to be built beneath the old depot and the adjoining land and streets. The plan for this, dated June 1, 1899, was adopted by the Board of Directors of the company but was not authorized for construction nor made public until after the fatal train collision in the Park Avenue tunnel on January 8, 1902. It was then laid before the Board of Railroad Commissioners of the state on the 23d of that month as offering a means of minimizing the smoke evil and increasing the capacity of the terminal. Before this date the writer, in August, 1901, had retained Bion J. Arnold, M. Am. Soc. C. E., to study the feasibility of handling heavy through trains by electricity between Mott Haven and the terminal, and his reports in February, 1902, were in the affirmative.

Meanwhile, the press of the city, particularly in the summer of 1901, was excoriating the Board of Directors for having done nothing, and all kinds of queer plans were advanced in its columns, and even in technical quarters, such as the proposal that the 2-mile tunnel should be operated as a single block; that ventilating apparatus with high chimneys should be installed for the dissipation of products of combustion; that the solid walls between the center and side tunnels should be removed; as was in fact tried out for a short distance with anticipated unsatisfactory results; that compressed air should be substituted

[11] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1736.

for steam in the movement of trains; that the tunnel should be converted into an open cut; and that electric illumination should be installed in the tunnel, long since frowned upon by the enginemen who by experience had found it to be wanting.

Concept of an Entirely New Terminal Utilizing Air Rights.—As he studied various proposals for retaining the old station in the new project, the writer was not satisfied that this was the ideal solution. In particular it was evident to him that a lofty new building for badly needed additional railroad offices on the site of the Annex would be in unhappy contrast with the neighboring imitation stone depot of less stately height. Why not (he questioned himself) tear down the old building and train shed and in their place, and in the yard on the north, create a double-level, under-surface terminal on which to superimpose office quarters and revenue producing structures made possible by the intended use of electric motive power? With this arrangement would go the erection of an adjoining hotel on the Harlem road's vacant square on Vanderbilt and Madison avenues between 43d and 44th streets (see Fig. 8), and the incorporation of a realty company for acquiring land and operating the rentable facilities. The keynote in this plan was the utilization of air rights that hitherto were unenjoyable with steam locomotives requiring the open air, or great vaulting spaces, for the dissipation of their products of combustion. Thus from the air would be taken wealth with which to finance obligatory vast changes otherwise nonproductive. Obviously it was the thing to do.

Plan of Revenue Producing Terminal and Improved Approaches.—It was on March 19, 1903, that the writer, as vice-president of the New York Central Railroad system, laid before President William H. Newman the detail plans that were intended to give expression to the parent idea conveyed to him in the preceding December. The objects were the erection of a suitable fifty-seven-track, all-electric, double-level terminal with a suburban loop, and a future rapid-transit connection should one be found to be desirable; the utilization of air rights producing income sufficient to pay interest on the cost of the terminal, the depression of the yard with street crossings restored from 45th Street to 55th Street, inclusive, and the attendant electrification, four-tracking, and other improvements to far-reaching points, including the elimination of 44 grade-street and highway crossings; the erection of a hotel and other means of attracting travel and revenue; the creation of a new north and south elevated city artery circumscribing the station building, extending over 42d Street on the south and joined on the north to a broad "Court of Honor" or "Grand Central Park" (see Fig. 9) on a future second level over the intersecting cross streets between 45th and 48th streets; and ramps connecting car-floor-level train platforms with the concourses. For this plan, Reed and Stem of St. Paul, Minn., were the architects for the station building. The opinion was hazarded that, in addition to being financially self-supporting, this treatment of the problem would result in the trebling or quadrupling of the suburban service when electrified.

As of May 6, 1903, the estimated cost of the improvements, exclusive of the cost of additional lands and interest during construction, extending from 42d

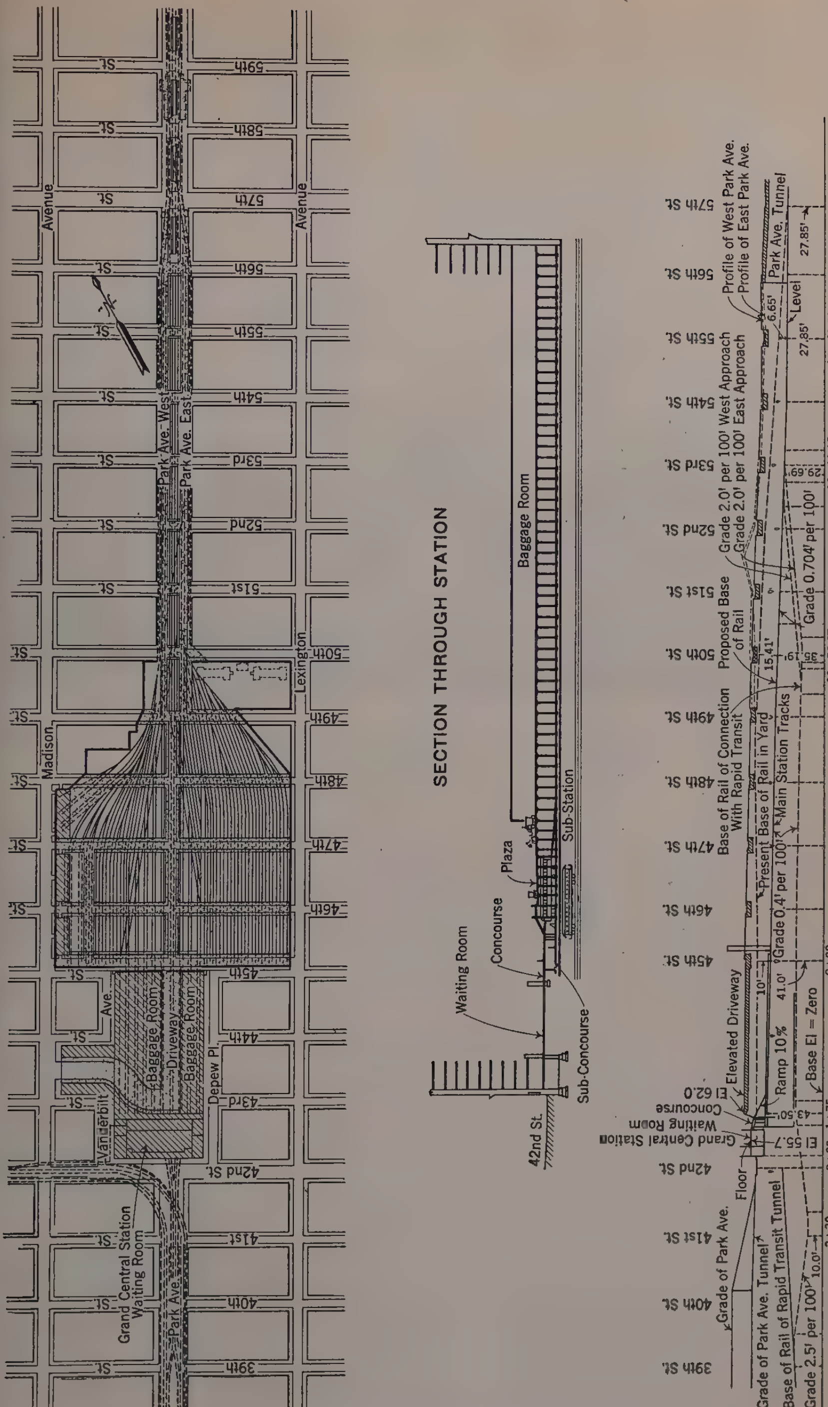

Fig. 8.—Proposed Transformation as of December 22, 1902, Predicated on the Utilization of Air Rights

Street to Croton-on-Hudson and North White Plains, 35 and 24 miles, respectively, including a hotel on the Harlem company's vacant square, was as follows:

Station	$ 8,500,000
Yard	6,750,000
Electrification	10,400,000
Four-tracking, etc.	7,810,000
Revenue producing facilities	10,000,000
Total	$43,460,000

Therefore, 20% of the total was assignable to the station proper, 15% to the yard, 24% to the electrification, 18% to the four-tracking and kindred work

FIG. 9.—PROPOSED TRANSFORMATION AS OF MARCH 19, 1903, LOOKING SOUTH, SHOWING PARK AVENUE RESTORED AND WIDENED TO CREATE A "COURT OF HONOR" OVER THE CROSS STREETS AND CONTINUED BY MEANS OF CIRCUMSCRIBING ELEVATED DRIVEWAYS AROUND THE STATION AND OVER 42D STREET

within the electric zone, and 23% to the superimposed and adjoining revenue producing facilities, including the hotel. The annual net earnings to be realized from rentals and privileges alone were placed at $1,450,000, or enough to yield in excess of 3% on the entire outlay, apart from what was to be gained from economies in operation and from increased travel. In other words, enough revenue was believed to be obtainable from the voluntary investment of 23% of the total cost to carry in large part the 77% that was unavoidable in the interest of the railroad companies and the public.

Approval of Plans by City and State and Commencement of Work.—On June 3, 1903, these plans, so much more comprehensive than those approved by the city in the preceding December, were laid before a special committee of its Board of Estimate and Apportionment, and were then declared by Mayor Seth Low to be proof of the thought he had had for some time that the changes which were to come about within a comparatively short time would entirely alter the complexion of the city. Nelson P. Lewis,[12] M. Am. Soc. C. E., chief engineer of the Board, said: "The plans impress me as providing perhaps the finest railway terminal station in the world. * * * I am deeply impressed with the magnanimous spirit of the company in planning things in a large and comprehensive way, without regard to cost."

[12] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1413.

In their entirety, including the electrification south of the Harlem River, the plans were formally approved by the city on June 19, 1903, under the provisions of the Laws of 1903, Chapter 425, that had been adopted by the state and accepted by the city on May 7 of that year; and the date of commencement of the work was fixed at July 1, 1903, with the understanding that it was to be completed within five years thereafter. Actual construction was started on track changes at 47th Street on July 18 and in full earnest on August 17, when the contractor for the yard depression, the O'Rourke Engineering Construction Company (of which John F. O'Rourke,[13] M. Am. Soc. C. E., was President), began the demolition of buildings on Park and Lexington avenues between 45th and 50th streets. Olaf Hoff,[14] M. Am. Soc. C. E., was engineer of structures, later succeeded by J. L. Holst, bridge engineer. A. B. Corthell,[15] M. Am. Soc. C. E., until well on in 1906, was resident engineer in charge of work in field and office, apart from that entrusted to the architects, and to Edwin B. Katté, electrical engineer, in respect to the electrification. It was planned that the yard excavation should be made in three successive "bites," each to be completed before another was undertaken, working westward from Lexington Avenue, so that the traffic of the three railroads using the terminal, by this time grown to more than 1,000 train and switching movements on a busy day, might continue without hindrance.

Concept of All-Electric Service Throughout Suburban Region.—It is now necessary to retrace steps for several months to the time when measures were undertaken to bring about the change in motive power. As an outcome of the writer's recommendation in March, 1902, that he should have the advice of experts in that respect, an Electric Traction Commission came into being on December 17 of that year, composed of Frank J. Sprague, Bion J. Arnold, and George Gibbs[16] (until September 5, 1905), Members, Am. Soc. C. E., as well as Arthur M. Waitt, the railroad's superintendent of motive power, shortly thereafter succeeded by J. F. Deems who took his place on the railroad. The writer made the fifth member and acted as chairman. Mr. Katté was made secretary and, as electrical engineer, was put in charge of the electrification work in field and office.

As expressed in his several reports to President Newman between 1901 and 1904, the writer's original concept was that the outward limits of the electric zone for both express and suburban service should be established at Croton-on-Hudson and North White Plains. This was despite the absence of a legal obligation to go farther north than the Harlem River, because it was his conviction that changing from steam to electric locomotives and vice versa within the city limits would seriously hamper traffic and would soon lead, by compulsion, to another banishment of steam still farther to the north. Equally strong was his belief that an all-electric frequent service in the suburban region would promote a marked increase in traffic, and a freindly attitude by the public, without which any corporation, railroad or otherwise, has "hard sledding." Nevertheless, there was a feeling by some in the Commission that electric

[13] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1707.

[14] For memoir, *loc. cit.*, Vol. 89 (1926), p. 1623.

[15] For memoir, *loc. cit.*, Vol. 88 (1925), p. 1374.

[16] For memoir, *loc. cit.*, Vol. 105 (1940), p. 1840.

locomotive service should not extend north of Mott Haven, because of the unpreparedness of the art for such a revolutionary change in the handling of extremely heavy, speedy, trunk-line trains. This was given voice in a letter dated June 8, 1903, to the writer from one of the Commissioners, who was supported in his position by another member. On October 31, 1903, a vote on the subject stood three to two in favor of the full extension, the writer casting the deciding vote. However, on the 3d day of the following month, the Commission unanimously decided that the plan as first promulgated should stand unchanged—namely that both multiple-unit suburban trains and express trains hauled by electric locomotives should be operated for the full distance between the Grand Central Terminal and the northerly termini at Croton-on-Hudson and North White Plains. Other important decisions reached at these and other meetings in 1903 were in favor of the use of direct current supplied to third-rail working conductors, and of vertical turbo-generators in the power stations. Dramatic conflicts of opinion developed between those weighted with the responsibility of making the enterprise work successfully and those on the outside who, without that responsibility, would see it done to their liking. Prominent among the critics was the late George Westinghouse, M. Am. Soc. C. E., who favored the adoption of alternating current supplied to overhead working conductors instead of direct current fed to a third rail, and who likewise favored the use of a horizontal turbo-generator. The A.C.–D.C. struggle later was to assume the dignity of a "celebrated case" comparable to the famous "battle of the gages" in England in bygone years. It gathered force when the New York, New Haven and Hartford Railroad, owning trackage rights south of Woodlawn, belatedly adopted the views of Mr. Westinghouse and found it necessary to plan an electric locomotive that could operate on both alternating and direct currents.

The deliberations of the Commission came to an end at their 108th meeting on October 5, 1906, after the initial electric zone had been completed and was ready for the running of the first electric train.

Intrusion of New Architects and Resultant Changes in Station Building Design.—It was not alone in the field of electrification that differences arose. After Reed and Stem had been accepted as the architects for the Grand Central Station building, the architectural firm of Warren and Wetmore in 1903 offered to the railroad company a plan of another type. Later the last named firm became a part of a new organization for designing the Grand Central buildings known as "Reed and Stem and Warren and Wetmore, Associated Architects," of which Charles A. Reed was named the executive, aided later by Mr. Corthell who had previously acted as resident engineer. The two new members of the firm did not look approvingly upon the revenue producing features of the original design and sought to change it to one more monumental in character, devoted solely to railroad purposes. In addition to disapproving the outside terraced driveways, they proposed that the treatment of the waiting room and concourse should be altered and the train room left unadorned. It must be confessed that the differing views in the end resulted in compromises that in some ways were praiseworthy, although the resulting abandonment of the elevated driveways around the station building and over the cross streets north

and south of it (in which the writer and Vice-President E. H. McHenry, M. Am. Soc. C. E., of the New Haven road joined in non-concurrence) was a grievous mistake that remained unrectified in full for many years. Happily, the station was so built that the elevated driveways could be added in the future, if and when it was deemed to be desirable. The intended high office building over the south end of the station on 42d Street also "fell by the wayside"; but provision was eventually made for its future construction around the concourse on the line of 43d Street. In addition to this the revenue producing features "held their own" on the street and lower levels of the station and now remain to enrich the coffers of the railroad companies. This is true to a remarkable degree where the supporting columns beneath the street surface came to be designed for superimposed 20-story revenue producers outside of the 6-story station area.

Continuing Evolution of Plans.—During the years that elapsed from the date of the first agreement with the City of New York, on June 19, 1903, to the fourth one dated July 8, 1907, the plans for the station and yard and the approaches within the electric zone were in constant flux owing to changing conditions; the opening of the purse of the railroad company for the purchase of additional land; the further requirements of public authorities; and the advancement of new ideas. At the station, as has been mentioned, the circumscribing elevated driveways were eliminated and the intended skyscraper frontage on

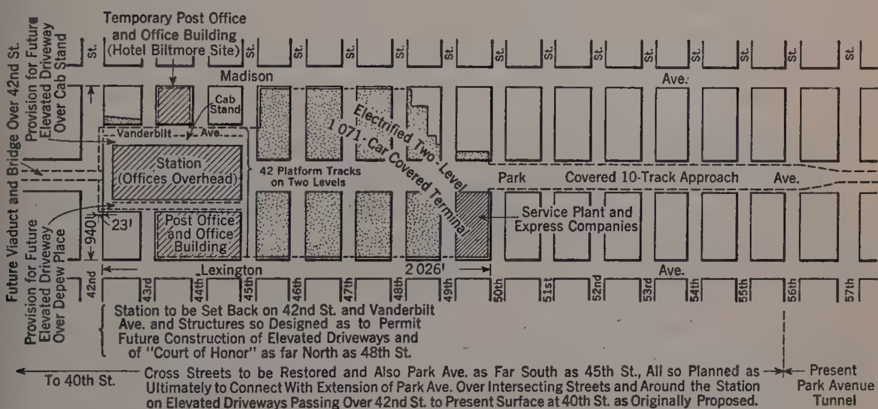

FIG. 10.—MAP OF PROPOSED TRANSFORMATION AS OF SEPTEMBER 30, 1907

42d Street abandoned in favor of a building of a uniformly moderate height covering the original site and also the adjoining area extending to Lexington Avenue between 43d and 45th streets. Likewise, the intended "Court of Honor" over the cross streets from 45th Street as far north as 48th Street, of a width equal to that of the main station building fronting on 45th Street, went into the discard (see Fig. 10). However, it was in part revived in after years when the restored circumscribing driveways were extended over 45th Street and thence on a descending grade to the present level of Park Avenue, of which the viaduct therein had been widened from 60 ft to 140 ft under the modified agreement with the city dated April 28, 1905.

Then, too, underground rights on that date were secured from the city in Vanderbilt Avenue and Depew Place, whereby the suburban station could be widened; the station front on 42d Street was set back 40 ft and on Vanderbilt Avenue 70 ft from the street lines; and the site of the 45th Street viaduct was moved southerly so as to be within the original lines of that thoroughfare. Rights in addition were secured from the city for building a low-level outlet sewer to the East River, 6 ft in diameter, situated in 46th Street, for the drainage of the far-down level of the terminal of which the lowest point was about 80 ft below the crown of the 45th Street viaduct.

Contract and Company Work and Quantities Involved at Terminal.—About seventy-three Grand Central Station and yard contracts, not including those for the electrification, were let from 1903 to 1907, inclusive. The most important contract (for excavation and masonry in the depression of the yard) continued in existence until May, 1907, when, by agreement, the work was taken over by the railroad and with notable success was continued by company forces under the direction of W. F. Jordan, Assoc. M. Am. Soc. C. E., reporting to the terminal engineer, then George A. Harwood,[17] M. Am. Soc. C. E., who had succeeded Mr. Corthell in 1906 and later became chief engineer of Electric Zone Improvements. The yard quantities to be handled from start to finish, according to the plans as they stood in 1907, approximated 3,000,000 cu yd of earth and rock excavation utilized in four-tracking and yard construction on the north, 260,000 cu yd of masonry, and 100,000 tons of steel. Twenty-five miles of pipes and sewers required change of location or replacement, and 27 miles of track called for laying and ballasting, together with their attendant 762-lever signaling and interlocking plant and the placing of passenger platforms along 6 miles of their length.

Capacity and Scope of Terminal.—At this time the area of both levels of the terminal embraced 66½ acres, or nearly three times that of the original depot and yard. Its plans provided for sixteen platforms served by sixteen tracks and a terminating double-track loop, with a possible rapid-transit connection on the suburban level; and fourteen platforms served by twenty-six stub tracks on the express level, terminating with one exception just north of the north line of 43d Street extended, thus totaling forty-two tracks serving thirty platforms on both levels. In addition to this, on the express level, were twenty-six non-platform tracks terminating at the concourse and at 45th Street, making in all fifty-two tracks on the latitude of 45th Street. Added to the sixteen in the suburban station this made a total of sixty-eight terminal tracks abreast on both levels, apart from miscellaneous storage and other tracks still farther north which brought the maximum number abreast to eighty-one. In all, the plans provided for the accommodation of 1,071 cars, or three times the capacity of the depot as it stood in prior years.

The sizes of the spaces intended for the use of travelers on the express level were 80 ft by 204 ft for the general waiting room and 160 ft by 465 ft for the main concourse, with a cab stand on the west having its outlet to Madison Avenue north of 43d Street. The frontage of the terminal on 42d Street measured 300 ft at the surface and 480 ft underground (see Fig. 11). The maxi-

[17] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 1700.

mum width of the upper level yard between Madison and Lexington avenues was 940 ft; that of the lower level, 475 ft.

In the plan the terminal yard on both levels ended on the north at 50th Street, 2,049 ft from the southerly limit of the terminal at 42d Street. The inner six approach tracks continued into the station south of 45th Street on a

FIG. 11.—PROPOSED TRANSFORMATION, 1907, VIEWED FROM THE SOUTHWEST; ELEVATED DRIVEWAYS ABANDONED TEMPORARILY

0.42% ascending gradient so as to aid in the stopping of trains, and the outer four, two on each side of Park Avenue, descended on a 2.16% gradient to the lower level, the entire ten main tracks fanning out from the four in the Park Avenue tunnel at 57th Street.

Status of Terminal and Associated Work at Close of 1907.—By the end of the formative period in the fall of 1907, the underpinning and side-walls on both sides of Park Avenue south of 57th Street, and the grading between them, had been finished sufficiently to permit the laying of four additional tracks, making eight in all, at the exit of the yard. The major portion of "Bite No. 1" on the easterly side of the terminal area along Lexington Avenue from 50th Street to 43d Street had been completed with its yard, substation, heating plant, express facilities, street viaducts, drainage sewer to the East River, and a temporary passenger station—all in actual use by the New York Central's electrified suburban service. With this had gone the demolition of approximately 200 buildings—a veritable slum clearance—including time-worn and smoke-stained churches, hospitals, and stores. The Annex had been demolished and excavation was in active progress down to the lower suburban level beneath Depew Place and north of it, and the terminal building was nearly completed on Lexington Avenue between 44th and 45th streets. The entire yard, old and new, had been electrified in connection with the use of a third rail of the protected under-control type, devised by the writer in collaboration with Mr. Sprague, and the signals and interlocking were adjusted to the new conditions under which the direction of traffic had been reversed from left-hand to right-hand running. All this work had been done while the daily train service, apart from switching movements, aggregating several hundred in number, had been steadily increasing from about 500 regular trains in 1903 to more than 600 in 1906.

On Madison Avenue, between 43d and 44th streets, a temporary post office and office building had been erected on what was to be the site of the Biltmore Hotel. To facilitate the acquiring of real estate and the handling of rentals, a subsidiary company had been organized, as had been suggested in the parent letter of December 22, 1902. In area available for train operation, the original of a million square feet had been increased to nearly a million and a half, and the car capacity from 374 to 519 cars, or 40%. Moreover, through the substitution of self-moving, multiple-unit electric cars for steam locomotive-hauled trains in the suburban service, after 1906, the total number of train movements in the terminal had been reduced approximately 30%.

In short, all was in order for the continuance of the terminal work as planned, awaiting only a final decision as to the type of station to be built south of 45th Street between Vanderbilt Avenue and Depew Place, and the possible purchase of the easterly portion of the block on the west side of Vanderbilt Avenue between 44th and 45th streets and a full block on 42d Street between Depew Place and Lexington Avenue, which later became the site of the Hotel Commodore, for a further enlargement of the two track levels and the introduction of an additional loop.

Exterior to the terminal the program was well advanced. Four-tracking on the Hudson Division, except at the Spuyten Duyvil cut, was in full use as far as the largely completed new steam and electric terminal at Harmon, N. Y. (South Croton). Two additional main tracks, making six in all, had been laid from Spuyten Duyvil to Ludlow, N. Y., at the south end of Yonkers. Four-tracking on the Harlem Division had been pushed on from Woodlawn Junction to Wakefield, N. Y., just south of Mount Vernon, N. Y., and right of way had been purchased, or nearly so, for its extension to North White Plains where a combined steam and electric terminal had been practically finished, barring the intended loop. With this program, in accordance with the original policy recommended by the writer that the elimination of all grade-street and highway crossings must precede electrification, the raising of tracks for more than 2 miles was in progress in Yonkers, with which was to go the building of three stations and a large freight yard. Similarly, the Marble Hill (N. Y.) Cutoff had been completed with its avoidance of many street crossings and a large saving in distance and curvature. Likewise, there were many other changes of alinement, and the new over and under street crossings with attendant stations on the Hudson Division were already in use or, in a few instances, awaiting an agreement as to plans. On the Harlem Division the plans for two major changes of line were completed, one at Mount Vernon and the other at White Plains, whereby grade-street crossings were to be avoided and sorely needed new passenger and freight facilities provided. At many other places on that division the elimination work with new stations had been completed or was well in hand. All elimination work, and in consequence the electrification on both divisions, north of the limits of New York City, had been seriously delayed by reason of unavoidably long negotiations with the communities affected and, above all, by the inexcusable previous neglect of the state authorities to act promptly on the plans presented to them for approval. Within the City of New York, not only had the main-line work of this nature been completed at

Marble Hill and at the three street crossings immediately south of it, and at Woodlawn on the Harlem Division, but also the work on the Port Morris Branch where a 1,773-ft double-track tunnel had been built under St. Mary's Park in conjunction with many overhead street bridges and the remodeling of its waterfront terminal.

One important feature in the rounding out of the new program was the separation of track grade crossings at the junction of the four-track main line of the Hudson and Harlem divisions at Mott Haven. Plans for this program, with a new station at 149th Street and a loop just south of it, were prepared and the necessary additional right of way purchased, with the idea that the suburban service, turned there as might be found desirable, would be operated at short intervals with varying lengths of trains to promote the growth of Westchester County, New York, and incidentally build up the Bronx as a satellite community to relieve future congestion in Manhattan. It seemed that such a course would accord with sound city planning and add to the safety of operation. Although it had then been authorized by the company, the plan has not been carried out in full, due, it is said, to financial and operating considerations.

Turning now to the subject of electrification, which has been treated in detail by the writer in a previous paper,[18] the two power stations had long since been completed in the fall of 1907, as had also the necessary substations, transmission lines, third-rail working conductors, repair facilities, signaling and interlocking, reversal of traffic to right-hand running, equipment consisting of electric locomotives and cars, standard plans, operating rules and instructions, trains schedules on which to base calculations for the future needs, and temporary electric and steam terminals at High Bridge, N. Y., 7 miles out on the Hudson Division, and at Wakefield 13 miles out on the Harlem Division. Beyond the temporary termini it was not then possible to extend the electric service for which the work was well in hand or completed, because of the delay in the approval of the plans for the grade-crossing eliminations to which reference has been made. Nevertheless, everything was in order to make the final touches to the electrification in the outer reaches of the electric zone as soon as these obstacles to safe operation were removed.

The time to test the new motive power, and the readiness of the Grand Central Terminal to handle it, came on September 30, 1906, when the first electric train was operated into the station with complete success. It was not until December 11, 1906, however, that regular electric service was inaugurated on a small scale, followed by its gradual expansion until, on July 1, 1907, the situation stood as shown in Table 1. In addition, there were the movements of construction trains which went to swell the total, and of switching locomotives still of the steam variety. So far as the New York Central company was concerned, under its contract with the City of New York, the use of steam locomotives in the Park Avenue tunnel was practically ended (96%) a year ahead of the agreed time, July 1, 1908. The delay in the conversion of the New Haven's service was due to its belatedness in its own territory beyond

[18] "The Electrification of the Suburban Zone of the New York Central and Hudson River Railroad in the Vicinity of New York City," by William J. Wilgus, *Transactions*, Am. Soc. C. E., Vol. LXI, December, 1908, p. 73.

Woodlawn Junction. Not until August 25, 1907, did it become possible to put the reversal of traffic from left-hand to right-hand running into effect.

It is by no means pleasing to record that while the change was in progress an electric train was derailed with fatal results on a curve near Woodlawn on

TABLE 1.—Regular Electric Train Service as of July 1, 1907

Description	Number of Trains			Percentages of electric trains
	Steam	Electric	Total	
(1)	(2)	(3)	(4)	(5)
Scheduled trains, New York Central and Hudson River Railroad	0	240	240	100
Shop trains to and from Mott Haven Yard	15	99	114	87
Subtotal	15	339	354	96
Scheduled trains, New York, New Haven and Hartford Railroad	122	30	152	20
Total	137	369	506	73

February 16, 1907. Bearing, as he did, the responsibility for having instituted and directed the Grand Central and other improvements in the electric zone, the writer very naturally was called upon by President Newman to defend the electric installation from the charge or suspicion that it was the cause of the wreck. This was successfully done to the satisfaction of Mr. Newman, W. C. Brown, the senior vice-president in charge of operation, the city authorities, the technical press, and the Board of Railroad Commissioners of the state.

In the execution of the work some ninety-seven contracts were let for the electrification, two of unprecedented size for signaling and interlocking, and twelve for electric zone improvements, in addition to the seventy-three at the Grand Central Terminal of which mention has been made. In all, as will be seen, some one hundred and eighty-four contracts were let for the work situated between 42d Street and the northerly termini of the region affected. In addition to this was the vast amount of work planned in detail under the direction of Mr. Harwood and V. Spangberg, designing engineers, and Mr. Katté, electrical engineer, and performed by company forces under the direction of George A. Berry and L. H. Byam reporting successively to the late Henning Fernstrom, M. Am. Soc. C. E., chief engineer, and W. H. Knowlton, principal assistant engineer, outside of the Grand Central zone, and under Mr. Jordan and James C. Irwin,[19] M. Am. Soc. C. E., within it. This included the installation of transmission lines and third rail; grading, laying, ballasting, and bonding tracks; digging and lining tunnels; erecting buildings, retaining walls, bridges, and fences; measures for reversing the direction of traffic and other signal work; snow and wrecking equipment; motive power facilities; and the disposal of excavated material by train from the Grand Central Terminal to outlying points.

The entire work by contract and by company forces had to be done in conjunction with the uninterrupted operation of an extremely heavy express

[19] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 105 (1940), p. 1864.

and suburban service centering at the Grand Central Terminal, where the number of train and switching movements, as has been explained, amounted to as much as 1,000 on busy days.

The dovetailing of the work in general was done through a Construction Committee which held in all 86 meetings between February 20, 1905, and October 31, 1906. On it were the heads of the various activities, including, at different times, Messrs. Fernstrom, Reed, Knowlton, Katté, Corthell, Harwood, Berry, Elliott, Ames, Hoff, Holst, and Spangberg of the engineering department, and A. T. Hardin and Ira A. McCormack of the operating department, the writer acting as chairman.

Estimated Cost of the Enterprise.—By the close of the formative period the estimated cost of the enterprise within the electric zone, terminating at 42d Street, as shown in Table 2, had increased from the original $43,460,000 to

TABLE 2.—Estimated Costs Exclusive of Interest

Description	Original Estimate, May 6, 1903		Modified Estimate, November 3, 1906		Increase	
	$	%	$	%	$	%
Grand Central Station (Railroad)	8,500,000	20	14,000,000	19	5,500,000	65
Grand Central yard	6,750,000	15	16,500,000	23	9,750,000	144
Electrification	10,400,000	24	14,500,000	20	4,100,000	39
Four-tracking, etc.	7,810,000	18	17,000,000	24	9,190,000	118
Revenue producers (approximately)	10,000,000	23	10,000,000	14	0	0
Total	$43,460,000	100	$72,000,000	100	$28,540,000	66

$72,000,000, exclusive of additional land purchases at the terminal and at the Marble Hill Cutoff, revenue producing adjuncts other than the projected Biltmore Hotel and space in the terminal building itself, and interest on money during construction.

It will be seen that the estimated cost of the combined railroad station and yard had increased in its proportion of the total for the entire project from 35% to 42%; that of the electrification had fallen from 24% to 20%; that of the four-tracking, with which went the elimination of grade crossings and rectifications of line and new stations, had increased from 18% to 24%; and that of the revenue producers had decreased from 23% to 14%. The latter did not include an estimated expenditure of many times as much for the tremendous growth of superimposed structures that ultimately was to follow—a growth that in time was to reach $120,000,000 in contrast with the modest beginning of $10,000,000. In all an investment of some $185,000,000 for improvements in and above the terminal and along its approaches, exclusive of interest during construction and land, was to result from the parent thought that by such means vast wealth was to be mined from the air as truly as from "the reefs of the Rand."

Grand Central Terminal Transformation—Concluding Period from 1908 to 1913

Partial Reversion to Elevated Driveway Plan.—It is a pleasure for the writer to record that the circumscribing elevated driveway plan, with its southerly

extension over 42d Street, came to life again in 1909, although only the westerly portion was built then, with a short descending grade to reach the level of 45th Street. The latter provision had been made in the original plan of March, 1903, so that the "Court-of-Honor" idea later might be put into effect, should it be concluded that the Park Avenue overhead crossings of 45th Street and the streets beyond were worthy of adoption.

Expansion of Track Layouts, Added Loop, and Station Rearrangements.—The subsequent purchase of additional land on Vanderbilt Avenue and on 42d Street between Depew Place and Lexington Avenue, to which reference has been made, led to the adoption of a plan for an additional loop, two storied, in 1909. At the time, this was particularly desirable for the use of the New York, New Haven and Hartford's locomotive-drawn suburban trains which, unlike the New York Central's multiple-unit trains, could not be properly handled in the lower level with its short-radius loop.

Moreover, this additional purchase, and that of property on the west side of Park Avenue between 48th and 51st streets, made it possible to plan an advancement of the throat of the west yards on both levels two blocks north to 52d Street, and in the station the expansion westerly resulted in an increase in platform space. Accompanying these changes in plan were others for the rearrangement of the approach tracks on the westerly side of Park Avenue and at their entrance at 57th Street, together with the steepening of the gradients to and from the lower level from 2.16% to 3% on the two descending tracks and to 2.7% on those ascending.

The northerly advance of the yard throats made it possible to do likewise with the southern end of the stub tracks on the upper level, thereby facilitating the rearrangement of the plans for the wai'ing room and concourse, and other changes in the station layout, in accordance with the final understanding reached by the presidents of the three railroads as provided for in the amended Triparte Agreement of which mention has been made. The results of these modifications are shown in Table 3 in which the data obtained in the original Grand Central Station of 1899 are compared with those applicable to the plan as it stood at the end of the formative period in 1907, and with those actually in being in 1925.

Thus, it will be seen that an addition of 19% to the terminal area as it was planned in 1907 made it possible further to increase the track mileage 23% and the platform car capacity 32%, although the total car capacity remained about the same. Compared with the original station of 1899, the capacity of the new terminal, in round terms, thus may be said to have been more than trebled. The area of the station building itself had not been increased greatly, but its facilities, measured by square feet, under both the plans of 1907 and what followed, had been quadrupled. The principal change made in their final proportioning was the decrease in the surface dimensions of the waiting room and its accessories, and of the concourses, as compared with what was proposed during the formative period.

Another reversion to the original recommendations of the writer, in addition to that of the elevated driveways, and an equally happy one, was the abolition of gas for car lighting on express trains in the terminal. The attempt had been

made during the formative period to bring this about in favor of electric lighting in all cars, express as well as suburban; but there had been opposition in some directions which long postponed what was clearly in the interest of safety.

TABLE 3.—COMPARISON OF COSTS

Description	Original station, 1899[c]	Grand Central Terminal[d]					
		As planned, January 1, 1908			As built, March 28, 1925		
		S	*E*	*T*	*S*	*E*	*T*
Area of terminal (acres)	23.24	24.22	42.29	66.51	31.21	47.89	79.10
Length of tracks (miles)	10.84	9.81	17.14	26.95	13.79	19.30	33.09
Number of Tracks:							
Abreast (maximum)	....	29	52	81	37	41	78
Against platforms	19	16	26	42	17	33	50
Number of platforms	....	16	14	30	14	17	31
Car Capacity (Number of Cars):							
Platform tracks	214	171	252	423	167	392	559
All tracks	366	389	682	1,071	502	629	1,131
Area of station building (acres)[a]	5.90	....	....	8.10	....	....	6.55
Area of Station Facilities (Sq Ft):							
Waiting room and accessories	19,922	....	....	103,295	....	....	66,411
Concourses	14,814	....	....	157,140	....	....	80,844
Baggage rooms	33,315	....	....	71,400	....	....	57,501
Cab stand and drives	2,952	....	....	39,640	....	....	25,493
Offices[b]	129,500	....	....	475,000	....	....	503,025
Post office and railway mail	33,000	....	....	104,495	....	....	154,642
Total area of station facilities	233,503	....	....	950,970	....	....	887,916

[a] At the street level. [b] Exclusive of corridors and toilets and New York Central railway building of 1925. [c] No suburban level. [d] Column headings: S = suburban level; E = express level; and T = total, both levels.

Opening of the Transformed Terminal.—The new terminal in its final form, apart from developments that were still to come, was opened to the public on February 2, 1913, nearly fourteen years after its birth throes in 1899, and

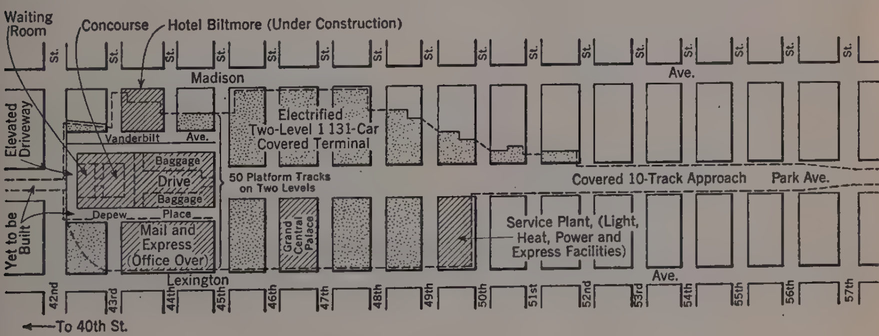

FIG. 12.—GRAND CENTRAL TERMINAL, OPENED FEBRUARY 2, 1913, WITH ELEVATED DRIVEWAYS PARTLY RESTORED

slightly over five years after its formative period had been brought to an end in the latter part of 1907. Referring to Fig. 12, the elevated driveways had been completed on the south and west sides of the station and held in abeyance

on the east side and north of 45th Street. The original idea was finally realized in full in 1927–1928, except that its northerly limit was fixed at 46th Street and its "Court of Honor" narrowed to the width of Park Avenue—140 ft. Provision for a future high office building was made on the line of 43d Street, surrounding the station concourse, instead of a little farther south as originally planned. In this concluding period the work had been directed by George W. Kittredge, Hon. M. Am. Soc. C. E., chief engineer of the New York Central system, assisted by the heads of the engineering and construction organization who, in large part, had been in charge of the work during the formative period. Those in responsible charge were Mr. Harwood, chief engineer of electric zone improvements, assisted by Walter L. Morse, M. Am. Soc. C. E., terminal engineer, W. J. Thornton, designing engineer, E. D. Sabine, resident engineer, and Mr. Jordan, manager of construction; also Mr. Katté, as chief engineer of electric traction, and Azel Ames, W. H. Elliott, and H. S. Balliet, successively signal engineer. In cooperation with them was Mr. Reed, executive of the associated architects until his death in 1911, after which Warren and Wetmore took his place. Mr. Harwood later was to become a vice-president of the company, as was Richard E. Dougherty, M. Am. Soc. C. E., whose early years in his profession were spent on the Grand Central Terminal transformation.

Completed Grand Central Terminal and Approaches from 1913 to 1939

Full Reversion to Elevated Driveway Plan.—The elevated driveway at the station was, in the first instance, built on its westerly side only, with a grade connection with Park Avenue at 45th Street. Many years after this it was completed as had been planned on the easterly side as well, and both driveways

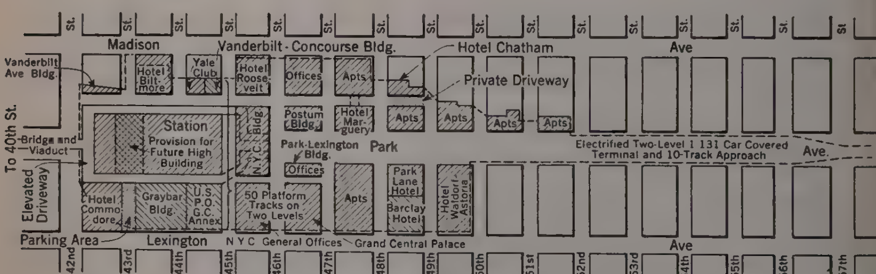

FIG. 13.—GRAND CENTRAL TERMINAL IN 1939

were then extended over 45th Street to a junction with Park Avenue at 46th Street. At least in part the plan of March, 1903, was thus made a reality. Fig. 13 shows an electrified, two-level, covered terminal, with superimposed revenue producers south of 52d Street and 50th Street. Also shown are the restoration of Park Avenue cross streets south of 56th Street as far south as 46th Street, and the elevated driveways over the restored 45th Street and around both sides of the station to a junction with the original Park Avenue at 40th Street. The dream of Mr. Reed in furtherance of the writer's concept of an elevated extension of Park Avenue southerly from 45th Street, ap-

proved at the time by Mayor Seth Low and Nelson P. Lewis, on behalf of the city, and by President Newman, on behalf of the railroad company, thus in principle had come true. It was to the combined genius of Mr. Harwood and Ira A. Place, and of Amos Schaeffer,[20] M. Am. Soc. C. E., on behalf of the city, that fulfillment of the original driveway plan was finally brought about in 1927–1928, a quarter of a century after its inception and 19 years after work on its westerly section had been initiated from 45th Street to 40th Street.

Fig. 14.—View on Park Avenue, Looking South from 50th Street in 1939, Showing Revenue Producers Over the Terminal and the New York Central Building in the Background, Through Which Connection Is Made with Elevated Driveways Surrounding the Station Beyond (Compare with Figs. 3, 7, and 9)

Superimposed Civic Center.—It will be recalled that with the coming of electricity as a motive power, and the opportunity thereby presented for the enjoyment of air rights which until then necessarily had lain fallow, it was

[20] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1689.

proposed that buildings should be erected over the terminal that would produce revenue. In fact, the steel columns beneath had been designed sufficiently strong to serve that purpose. Gradually, in time, primarily under the guidance of Mr. Newman, this had been brought about to a degree that has far exceeded the fond expectations of the writer. Over the tracks have risen such world-known hotels as the Biltmore, the Commodore, the Roosevelt, the Waldorf-Astoria, the Park Lane, the Barclay, and the Chatham; such outstanding office structures as the Graybar and New York Central buildings and the overhead portion of the Grand Central Terminal buildings; such erections intended for special purposes as the Grand Central Palace, the Yale Club, and the U. S. Post Office buildings; and stately rows of apartment buildings of the highest class along Park and Lexington avenues as far north as 50th Street, and along Madison Avenue and Vanderbilt Avenue as far north as 48th Street and 49th Street, respectively (see Fig. 14). The Grand Central Zone has become a self-contained city clearly evident to the casual onlooker who little knows that beneath it are the terminal yards of two great railroad systems.

Approaches Within New York City.—The work on the approaches, as planned, had been practically completed during the formative period, excepting the Spuyten Duyvil rectification, never since undertaken, and the Mott Haven Improvement for which the land was purchased but the work ever since held in abeyance other than the laying of additional tracks, the widening and lengthening of the 149th Street cross viaduct, and the commencement of a passageway connecting with the adjoining rapid-transit express station at Mott Avenue. The Mott Haven project in one form or another perhaps will have its day, as the modern move in city planning in favor of satellite communities grows in strength, as the growing needs for relief at the Grand Central Terminal may prompt the stopping of a portion of the suburban service at an outer point such as this, and as the demands of the public for the stoppage of express trains in a borough of a million and a half population become more insistent.

Approaches in Westchester County.—On the Hudson Division the improvements as planned were completed in the instances where that had not already been done during the formative period; but on the Harlem Division the four-tracking was completed to Mount Vernon only. Beyond that the laying of the intended additional two tracks and the construction of the wye at North White Plains have not been completed; but the new stations and grade crossing eliminations in that territory have been finished with due regard for the future four-tracking as planned. It is to be expected that the continued remarkable growth of Westchester County, particularly by reason of its character, will in time call for the rounding out of the improvements as originally designed.

Beneficial Results

Railroad Blessings.—To the railroads using the transformed terminal and its approaches have come numerous blessings. Safety of train operation through the abolition of smoke and gas in the Park Avenue tunnel; adequacy of facilities for the efficient handling of a constantly increasing passenger service; the minimizing of "dead" train haulage to and from Mott Haven and of switch-

ing in the terminal yard; economy of operation as demonstrated in the writer's paper[18] of which mention has been made and set forth in reports made to Mr. Newman in August and September, 1907; revenue gained on a vast scale from superimposed buildings in the Grand Central zone; the inducement of travel over the lines of the railways having their terminal in the heart of a civic center practically under one roof; the advertising value of an electrically operated double-level underground terminal centered in a capacious station favorably known to the entire world—all these well may be termed blessings.

To these, of course, should be added the growth in the number of passengers, made possible by the expansion and convenience of the improved terminal and its approaches, and induced by the removal of deterrents incident to the use of steam motive power. Evidence of this is shown in Table 4.

TABLE 4.—Passengers in and out of the Grand Central Terminal (in Thousands)

Description	1906		1930		Percentage of Increase	
	Commuters	All	Commuters	All	Commuters	All
Hudson Division	3,065	5,829	9,913	14,605	223	151
Harlem Division	3,570	4,905	16,351	19,160	358	291
Total New York Central and Hudson River Railroad[a]	6,635	10,734	26,264	33,765	296	215
New York, New Haven and Hartford Railroad	3,177	8,305	9,444	16,878	197	103
Grand total	9,812	19,039	35,708	50,643	264	166

[a] Including New York and Putnam and 125th Street stations.

In a word, the traffic of the New York Central road in 24 years had quadrupled in its commuter service and more than trebled in its combined commuter and express service, whereas that of the New York, New Haven and Hartford had trebled and doubled, respectively. For both roads in 1930 there was almost a quadrupling of the number of passengers that had been moved in commuter trains in 1906 and something less than three times the number handled in the trains of both services. The new electrified terminal with treble the car capacity of its steam predecessor, or quadruple the capacity if weight is given to the diminished need for switching through the use of electric motive power, had, therefore, fully measured up to the needs of the occasion in 1930. Taken all in all, so far as the writer is aware, the enormous expenditures made on the terminal and its approaches have been fully justified by results, judged from the viewpoint of the railroads concerned.

Public Blessings.—Steam railroads almost invariably have a depreciating effect on neighboring property, especially in urban and suburban communities where their smoke, gas, cinders, noise, and unsightliness in the open discourage civic development and are productive of shabby surroundings or even slums. There was no more striking evidence of this than in the vicinity of the original Grand Central Depot yard and therefrom northerly along Park Avenue as far

as 96th Street. If proof of this were needed, resort may be had to a comparison of the assessed values of real estate in 1904 before the intent to transform the terminal had become widely known and in 1930 when completed (see Table 5).

It will be seen that the increase in Manhattan as a whole was 175% (174.7% to be exact) and in Section 5, exclusive of the Grand Central zone and beyond, 168%. It is fair to assume that had not the Grand Central improvements with their restoration of the cross streets and Park Avenue been effected the

TABLE 5.—ASSESSED VALUES, IN MILLIONS OF DOLLARS

Item	Regions	1904	1930	Increase	
				$	%
1	New York City	5,015	19,717	14,702	293
2	Borough of Manhattan	3,677	10,102	6,425	175
	Section 5 (Block 1,280):				
3	Within the Grand Central zone[a]	700	2,427	1,727	247
4	Beyond the Grand Central zone[b]	432	1,159	727	168
	The Grand Central Zone and Beyond:[c]				
5	Actual	268	1,268	1,000	374
6	Computed at the same rate of increase as Manhattan (174.7%)	268	735	467	175
7	Increase attributable to the transformation	0	533	533	...
	Westchester County:				
8	Actual	193	1,758	1,565	813
9	Computed at the same rate of increase as New York State as a whole	193	796	603	313
10	Increase attributable to causes arising from the transformation	0	962	962	...

[a] Between 40th and 96th streets from the East River to Sixth Avenue as far north as 59th Street; then to Fifth Avenue as far north as 96th Street. [b] Exterior to the Grand Central zone and beyond, to 96th Street. [c] Between 42d Street and 96th Street and between Madison Avenue and Lexington Avenue (assembled through the kindness of Harry P. Moran, Stanley Skarvan, and Frank Klopf, of the U. S. Works Progress Administration).

increase in assessed values in the section of which they are a part would have been no greater than its remainder, namely 168%. The chances are that it would have been less because of the continued depressive effect on property facing a smoky, noisy, unsightly terminal and the open craters along Park Avenue from whence smoke, gas, and cinders were belched. However, on the more conservative assumption that, were it not for the terminal improvements, the situation would have corresponded to what took place in the borough as a whole, the increase in assessed values attributable to the transformation may be stated as shown by items 5, 6, and 7, Table 5.

The result to the people of New York in enhanced assessable values because of the Grand Central improvements, therefore, may be taken at something in excess of $500,000,000 and yielding in taxes at the current rate of 2.7% more than $14,000,000 per annum. The incalculable indirect effect of the change on more distant property has also been very great.

In a similar manner it is possible to view the situation in Westchester County as shown by items 8, 9, and 10, Table 5. It would seem fair to demonstrate in this way that in Westchester County there has been an increase in assessed values of nearly $1,000,000,000 in excess of what would have occurred if the

change of motive power and other railroad improvements had not been made. The elimination of steam locomotive nuisances, and of grade-street crossings with their accompaniment of danger and delay, certainly had a pronounced effect on the attractiveness of the region as an abode for city workers.

With this went a more frequent train service, the freeing of passengers from terror in the passage of the Park Avenue tunnel, and the convenience and protection from the weather that has come from affording a means for circulation by the travelers under cover in the midst of a new civic center supplied with adjacent express stations of the city's rapid-transit systems.

If further proof of the beneficial effect of the change of motive power is required, it may be found in the marked difference between growths of population, travel, and assessed values of real estate in the commuter regions in which that change has come about north and east of New York City, and in the commuter region west of the Hudson River where such a change had not been effected in 1930.

TABLE 6.—Comparison of Percentage Increases—Communities Served by Steam Trains and by Electric Trains

Item	Description	Steam[a]	Electricity[b]	
			Westchester County	Long Island
1	Population: 1930 over 1900.........	142	183	249
	Passengers Handled:			
2	1930 over 1911................	69	....	292
3	1930 over 1906................	....	166	....
	Assessed Value of Real Estate:			
4	1930 over 1911................	....	440	597
5	1930 over 1914................	161	....	....

[a] Percentage increases in the steam-operated region west of the Hudson River. [b] Percentage increases in the electrically operated regions north and east of New York City.

From Table 6 it will be seen that in New Jersey and New York west of the Hudson River the increase in assessed valuations was only a fraction of that in Westchester County (161% against 440%) and the disparity is still greater when the comparison is made with Long Island—161% against 597%. The comparisons, of course, are not exact, as the earlier dates differ due to an absence of obtainable data for corresponding years; but they are believed to be sufficiently accurate to warrant the drawing of general conclusions for the purpose of illustration. Unquestionably, the pronounced increase in property values in Westchester County is largely, if not entirely, to be ascribed to the impulse that had its start in the change of motive power and associated improvements. Had there been no change, it is fair to assume that the outcome would have been no more happy than it was on the westerly side of the Hudson River.

Then, too, it is believed that the change of motive power and other improvements in the railroad service in Westchester County was the spark that led to the initiation of parkways and parks in that region from which gradually

came forth a system of pleasure thoroughfares and recreational facilities of which there is none more beautiful in the United States.

Assuming that the new Grand Central Terminal and its associated changes for the better were the primary cause of the phenomenal increase in assessed real-estate values in Westchester County, the resulting indirect benefit to the public at the current tax rate of 2.706% is in excess of $26,000,000 per annum.

Résumé and Conclusion

The Project in Perspective.—In this paper the attempt has been made to portray a scene in which the parent reasons for doing things, woven on a background of early history, would invest the rounded image with proportions which, seen from a distance, may not be distorted.

In the mind of Commodore Vanderbilt was born the idea of a new terminal on a 40-yr-old ramshackle entrance line in New York City, to which was to be brought the traffic of a great system of railroads under his control. Given life by him, the resulting Grand Central Depot, with its approaches, continued to function through three succeeding epochs of reconstruction and enlargement, until after twenty-eight years, in 1899, the use of steam locomotives in and approaching it had become insufferable and its design and capacity had become unsuited for the demands that had come upon it; sixty-eight years had now elapsed since the owning company had made its first bow as a horse railroad in 1831.

The inescapable substitution of electricity for steam as a motive power (experimental though it was for trunk-line service), combined with the idea, born in 1902, that revenue plucked from the air might be used to finance its tremendous cost, had its fruit in the inception of a new Grand Central Terminal with the electric service extended to the limits of the suburban zone. Following this step came ten years spent in development of plans, the construction of the project, and the opening of the transformed terminal to public use on February 2, 1913.

Thenceforth the project was rounded out and blessings reaped which have their evidence in the multiplied travel and revenues enjoyed by the New York Central and New York, New Haven and Hartford systems; in the replacement of slums in the heart of the City of New York by a remarkable civic center; and the giving of an impulse to the development of Westchester County, which, in the form of taxes, is bringing some $40,000,000 per annum to the public purse.

The Future.—Attention has been directed to the trebling of the number of passengers handled in the terminal since 1906, and to its capacity which likewise has been more than trebled. It would seem, therefore, that the time is not far distant when, as was done forty years ago (1899), thought must be given to the taking of measures for handling a further increase of traffic and for satisfying public demands. What may have been done in this direction the writer is unaware. The installation of a satellite terminal at or near the Mott Haven yard in the Bronx would seem to be worthy of consideration as one way out; other suggestions are the provision of enlarged facilities in the Grand Central zone in connection with a new crosstown link to the Hudson Division

main line on the West Side, as was proposed by the writer during the formative period, or a new tunnel beneath the present viaduct and tunnel in Park Avenue, or in part beneath adjoining thoroughfares as has been suggested by the Regional Plan Association of which H. M. Lewis, M. Am. Soc. C. E., is chief engineer and secretary. It is to be hoped that a possible solution for this problem may have public discussion without waiting for the pressure of events such as made the railroad companies' lot so painful in the closing years of the nineteenth century.

Acknowledgments

To the New York Central System the writer owes his thanks for permission to deposit his voluminous duplicate company records in the New York Public Library, for certain of the photographs herein used, and for up-to-date plans of the terminal. It is perhaps needless to add a further expression of the writer's gratitude to those who were associated with him in the remodeling of the old steam-operated Grand Central Station and in the formative years of its transformation into its electrically operated successor which became known as the Grand Central Terminal; also to those of the Tax Department of New York City and of the U. S. Works Progress Administration who gave of their time in the assemblage of assessed values. The names of those who were more prominent in the carrying of responsibility in these respects are given in the body of the text; the names of the others as a rule are given in the technical press of the day.

The very nature of this paper has required a rather complete statement of the persons involved. Such mention, however, has been conservative and in full recognition of the Society's rule against discussing personalities in its technical papers. It is expected that discussers, likewise, will confine themselves to the technical problems, and not the persons, as the subject of their comments.

Copies of the manuscript in full, from which this paper has been prepared, including illustrations and laymen's drawings, may be consulted at the Engineering Societies Library[21] in New York, the New York Public Library, and the Congressional Library in Washington, D. C.

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[21] 29 West 39th Street, New York, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

EARTHQUAKE STRESSES IN THE SAN FRANCISCO-OAKLAND BAY BRIDGE

By Norman C. Raab,[1] M. Am. Soc. C. E., and Howard C. Wood,[2] Assoc. M. Am. Soc. C. E.

Synopsis

The combination of deep piers and long spans, of various types and degrees of rigidity, in the San Francisco-Oakland Bay Bridge, resulted in unusual problems in the calculation of earthquake stresses. This paper describes the assumptions which were made, typical examples of stress analysis, and the provisions incorporated to resist seismic forces. It was found that an acceleration equivalent to 10% of gravity produced stresses in the superstructure of the same order as those arising from the assumed wind loads. In general, very little additional material was required to resist earthquake forces. In the following, use has been made of data, including vibration measurements on the structure, not available at the time of the actual design. To such extent, this paper may be considered a review of the design.

Description of Bridge

The bridge, with its approaches, extends from San Francisco, Calif., to Oakland, Calif., a distance of 8¼ miles. It is naturally divided into the San Francisco Section, the West Bay Crossing, the Island Crossing, the East Bay Crossing, and the East Bay Approaches. The structures on the approaches and on the Island were such as to permit usual methods of seismic analysis and will receive no further comment herein.

The West Bay Crossing extends from the San Francisco Anchorage to Yerba Buena Island (see Fig. 1(*a*)) in San Francisco Bay. It consists of a twin suspension structure with a common central anchorage. The center spans are each 2,310 ft, and the side spans are each 1,160 ft in length. To secure better foundations, the San Francisco Anchorage was located 863 ft from the end of the suspended structure. This resulted in a comparatively long backstay cable which makes the west half of the suspension structure somewhat more

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **February 15, 1941**.

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flexible than the east half. The piers range in depth from 100 to 240 ft. At Pier 6, the mud line is 105 ft below low water. The strata through which the piers extend consist of about 20 ft of comparatively soft silt, underlain by consolidated clays and sands. All piers are founded on rock (Franciscan sandstone).

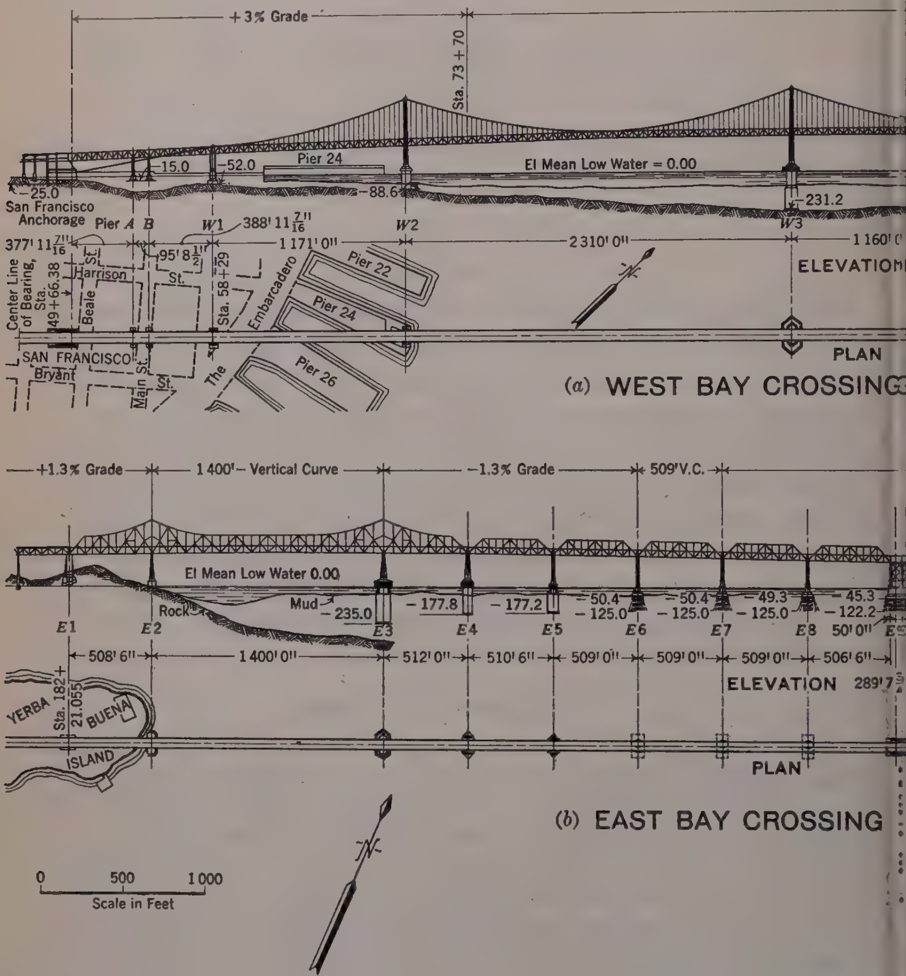
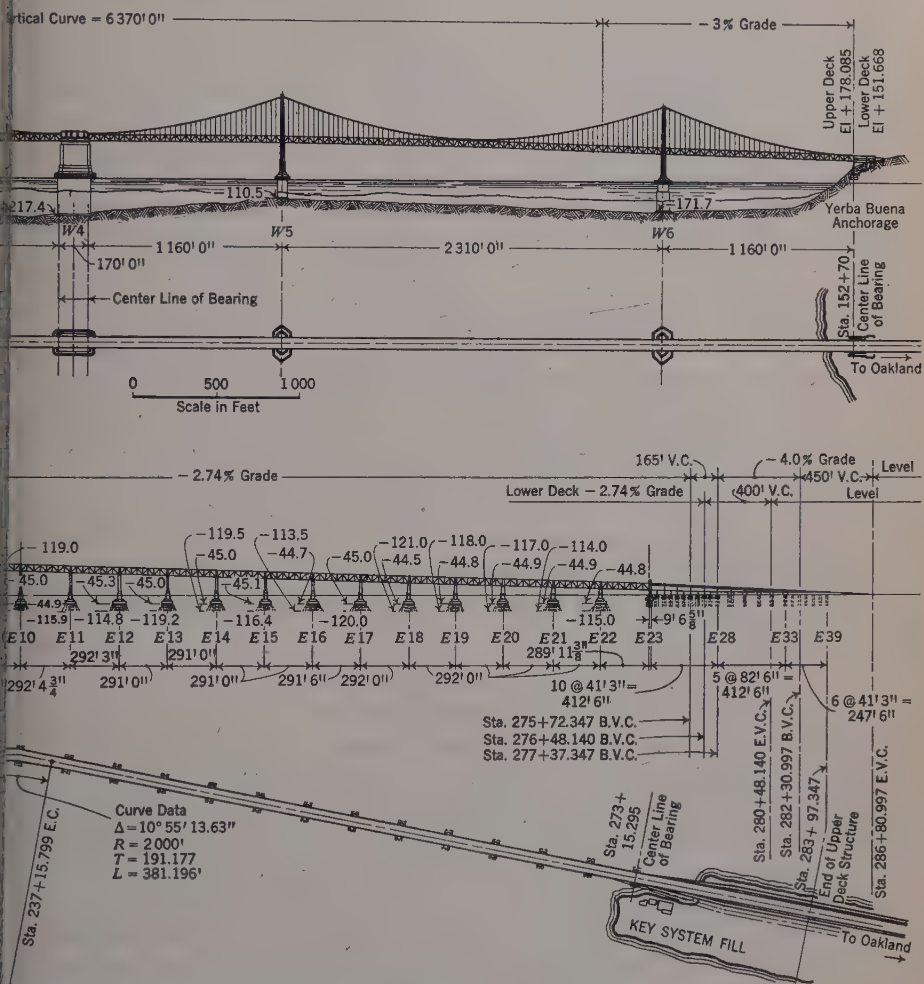


FIG. 1.—PLAN AND ELEVATIO

The East Bay Crossing (Fig. 1(*b*)) extends from the Island to the Oakland shore line. It consists of four 288-ft spans, a cantilever structure with a 1,400-ft center span, and two 508-ft anchor arms, five 504-ft and fourteen 288-ft spans.

To provide anchorage against longitudinal forces, the structure is anchored at piers YB-1, E-1, E-9, and E-17, with provision for expansion at piers YB-3, E-4, and E-11. At Pier E-4, provision is made for the expansion, arising from

temperature and stress, of nearly a mile of bridge. East of the Island, the rock slopes off very sharply. Pier E-2 is founded on rock at El. −40. Pier E-3 extends to a sand and gravel stratum at El. −240, and piers E-4 and E-5 are founded on similar strata at El. −170. The remaining piers were supported on piles although their bases were extended to comparatively solid sand strata.



WEST BAY AND EAST BAY CROSSINGS

GEOLOGY

San Francisco Bay lies on the surface of a long fault block or strip of the earth's crust trending roughly northwest and southeast (see Fig. 2). The block is bounded on the east by the Hayward fault, a nearly vertical fracture located along the west base of the Berkeley Hills and hence along the eastern margin of such East Bay cities as Berkeley and Oakland, Calif. Along the

west side of the block is the San Andreas fault, likewise a vertical break in the crust, and lying a few miles offshore from the mouth of the Golden Gate. The block is hence about 20 miles wide, its length is many times its width, and its depth is not less than 25 or 30 miles. The Bay only occupies a fraction of its width, the cities of San Francisco and Oakland resting upon unsubmerged parts of it. The bridge is located roughly one third of the distance from its eastern to its western side; the cantilever span is about 7 miles, and the suspension spans about 9 miles distant from the Hayward fault, and they are respectively about 12 and 10 miles from the San Andreas fault.

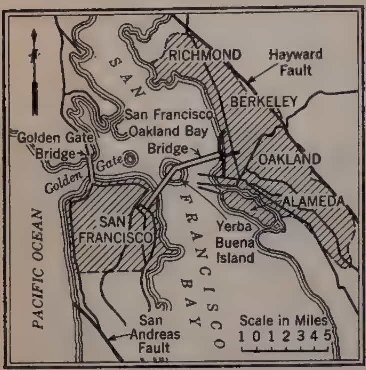


Fig. 2.—Location of Active Faults

Besides the San Andreas and Hayward faults bounding the block, many faults of less importance are known to traverse the interior of the block. The San Andreas and Hayward faults are known to be active, as evidenced by both strong earthquakes and numerous minor shocks which have occurred on them from time to time, and by characteristic fault topography developed along them by the successive movements in past centuries. The numerous minor fractures within the block exhibit no such topographic testimony of activity, nor are earthquake epicenters located on them instrumentally from time to time; hence, they are regarded by geologists as inactive or dead faults. Neither earthquakes nor relative dislocation of the ground on the two sides of these internal faults need be feared.

Earthquake Forces and Effects

The designers of the bridge reviewed the available data as to earthquake force and damage and formed the following opinions:

1. Material damage during earthquakes has been confined to structures of poor workmanship or to structures whose elements of greatest rigidity were not designed to transfer horizontal forces. Conversely, well-built structures, even if no definite allowance for seismic forces has been made in their design, have survived the most severe California earthquakes with no structural damage.

2. Japanese earthquakes appear to be of greater intensity than those in California; however, in Japan, structures designed for a horizontal acceleration of 10% of gravity have survived the heaviest earthquakes with no material structural damage. It appears safe to assume a smaller acceleration in designing against California earthquakes.

3. The maximum ground accelerations in Japan probably reach at least 50% and in California at least 25% of gravity.

4. It appears evident, therefore, that there must be some relieving factors not usually considered in a seismic design. These factors probably involve the mass and elasticity of the structure, its supports, and the period of the earth-

quake. It is a matter of observation that of two geometrically similar solids, the smaller is more likely to be overturned during an earthquake.

5. Although the ground motions during the violent phase of an earthquake are probably never truly harmonic, they may approach this condition for a limited number of vibrations.

6. The best information as to earthquake periods seems to be as follows:

(a) From the information now available, there is no conclusive evidence as to the existence of dominant ground periods, although certain periods may stand out more prominently than others.

(b) For the same character of ground, the shorter periods are damped out the more rapidly with increase in distance from the epicenter; hence, with increasing epicentral distances, the longer periods tend to prevail.

(c) In general, at equal epicentral distances, periods in alluvial soil are longer than in rock.

(d) At this particular location, considering the distance from the bridge to the San Andreas and Hayward faults and the fact that the piers are founded on rock or other solid strata, it appears probable that the period during the most violent phase of a strong earthquake would not exceed 0.7 sec.

(e) During the end phase of heavy shocks, almost pure simple harmonic motion, with a period of from 6 to 8 sec, may be expected at great distances from the epicenter. However, the information now available records no structural damage during such periods.

7. The evidence as to resonance and the dangers thereof is very conflicting. If resonance builds up to the extent assumed by some writers, the damage, during even a moderate earthquake, would be much greater than has been the experience. A possible explanation is that:

(a) A minor change in period such as may result from elastic yielding of foundations may be sufficient to destroy resonance.

(b) Close synchronism between the natural period of the structure and the period of forced vibration is required to establish resonance, especially in the higher modes.

8. In any vibratory motion, the maximum acceleration at any point exists only for an instant. In an elastic structure with a large percentage of its mass a considerable distance from the point of application of the earthquake motions, there will be a time lag before all parts of the structure are accelerated. Therefore, the maximum acceleration will not occur simultaneously in all parts of the structure.

9. Assuming resonant or near-resonant conditions, more or less damping is certain. There are no data sufficient to permit calculations of this effect.

Design Assumptions

Before proceeding with any calculations of stress, it was necessary to assume an hypothetical earthquake and the nature and intensity of the forces to which the structure would be subjected by such an earthquake and, also, in so far as possible, to analyze the resulting stresses. In many cases, the system is so complicated that a quantitative solution cannot be reached. For these cases, an attempt was made to assume forces higher than those that are believed probable.

Hypothetical Earthquake.—The earthquake forces were assumed as those resulting from a ground motion with a horizontal acceleration of 10% of gravity, a period of 1.5 sec, with a corresponding amplitude of 2.2 in. This is a longer period than is probable. It was chosen because, with other assumptions used in the design, it generally produced higher stresses than the same acceleration with a shorter period. However, the possibility of resonance at other periods was investigated.

Effect of Surrounding Water and Silt.—As the pier is moved by the rock on which it rests, the pier in turn displaces the surrounding water and, possibly, part of the surrounding silt.

For the analysis of these effects, studies were made of the paper by H. M. Westergaard, M. Am. Soc. C. E., on "Water Pressures on Dams During Earthquakes,"[3] and of the discussions of this paper. On the basis of the concept of "apparent mass," the equation in the discussion by Theodor von Kármán, M. Am. Soc. C. E., was used to determine the horizontal dimension of this mass. It was assumed that the boundary of the apparent mass was not affected below the mud line by the different specific weight encountered there. Equations for unit pressures due to earthquake were then developed, together with expressions for the shears and moments resulting therefrom. In the case of a bridge pier, the water has two paths of escape; vertically (which is the only possible one in the case of a dam), and around the ends. The dam analogy is very severe, since the actual movements would be a combination of the two paths. Because water is present on both sides of the pier, the acting forces were doubled; that is, "apparent masses" were assumed to exist simultaneously on opposite faces of the pier.

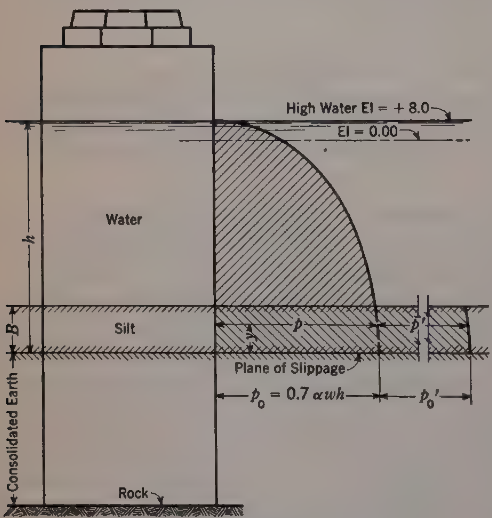


FIG. 3.—WATER AND SILT PRESSURES ACTING ON PIERS

In locations, such as tidal flats, there is evidence that during earthquakes the surface mud behaves as a viscous liquid and does not move as a unit with the underlying strata; that is, a slippage plane exists. The thickness of this moving mud is unknown, but considerations based on friction indicate that it is not great. The friction resulting from the weight of 60 ft of water above the mud may preclude the existence of a slippage plane. However, assuming such a plane, it is difficult to evaluate the force required to displace this mud. To be conservative, a slippage plane was assumed to exist at the top of the firm clays and sands, giving a moving mud layer in the general order of 20 ft.

[3] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 418.

For purposes of stress analysis, it was assumed that this material acted as a liquid with a specific weight of 100 lb per cu ft and, further, that it might be acting in a phase 180° from the movement of the pier. Here again the forces were doubled, since mud is present on both sides of the pier. The forces acting on one face of a pier are indicated by Fig. 3. Let h = depth from water surface to plane of slippage; b = horizontal dimension of "apparent mass" at distance "y" above the plane of slippage. Then

$$b = 0.7\,(h^2 - y^2)^{0.5} \qquad (1)$$

Let p = unit pressure caused by earthquake, if the pier were surrounded to a depth h by water only; p' = additional unit pressure below the mud line; w = weight per unit volume of water; w' = weight per unit volume of silt; a = acceleration of the pier relative to that of the water (that is, the assumed earthquake acceleration); a' = acceleration of the pier relative to that of the mud or silt; $\alpha = \frac{a}{g}$; and $\alpha' = \frac{a'}{g}$.

The unit pressure on one face of the pier above the mud line becomes

$$p = \alpha\, w\, b = 0.7\, \alpha\, w\, (h^2 - y^2)^{0.5} \qquad (2)$$

Between the mud line and the plane of slippage, the unit pressure is

$$p + p' = \alpha'\, w'\, b \qquad (3)$$

From Eqs. 2 and 3,

$$p' = \alpha'\, w'\, b - \alpha\, w\, b \qquad (4)$$

$$\frac{p'}{p} = \frac{\alpha'\, w'}{\alpha\, w} - 1 = k \qquad (5)$$

and

$$p' = k\, p = 0.7\, \alpha\, w\, k\, (h^2 - y^2)^{0.5} \qquad (6)$$

At the plane of slippage, where $y = 0$,

$$p_0 = 0.7\, \alpha\, w\, h \qquad (7)$$

and

$$p_0' = 0.7\, k\, \alpha\, w\, h \qquad (8)$$

As stated, due to the presence of water on both faces of the pier, the total load per unit area was taken as equal to twice the pressures expressed by Eqs. 7 and 8. On this basis, the total shear resulting from the load p on two opposite faces of the pier is

$$V = 2 \int_y^h p\, dy = 1.4\, \alpha\, w \int_y^h (h^2 - y^2)^{0.5}\, dy \qquad (9)$$

When $y = 0$, Eq. 9 becomes

$$V_0 = 1.1\, \alpha\, w\, h^2 \qquad (10)$$

Similarly, the shear resulting from p' is

$$V' = 2 \int_y^B p'\, dy = 1.4\, k\, \alpha\, w \int_y^B (h^2 - y^2)^{0.5}\, dy \qquad (11)$$

in which B = depth of mud above the plane of slippage. When $y = 0$, Eq. 11 becomes

$$V_0' = 0.7\, k\, \alpha\, w \left[B\,(h^2 - B^2)^{0.5} + h \sin^{-1} \frac{B}{h} \right] \quad (12)$$

The total shear at any elevation is

$$V_T = V + V' \quad (13)$$

The moment resulting from p is

$$M = 2 \int_y^h p\, y\, dy = 1.4\, \alpha\, w \int_y^h (h^2 - y^2)^{0.5}\, y\, dy \quad (14)$$

When $y = 0$, Eq. 14 becomes

$$M_0 = 0.47\, \alpha\, w\, h^3 \quad (15)$$

The moment resulting from p' is

$$M' = 2 \int_y^B p'\, y\, dy = 1.4\, k\, \alpha\, w \int_y^B (h^2 - y^2)^{0.5}\, y\, dy \quad (16)$$

When $y = 0$, Eq. 16 becomes

$$M_0' = 0.47\, k\, \alpha\, w\, [h^3 - (h^2 - B^2)^{1.5}] \quad (17)$$

The total moment is

$$M_T = M + M' \quad (18)$$

Assuming no support from the consolidated earth (Fig. 3), the shear at any distance Z below the plane of slippage is

$$V_Z = V_0 + V_0' \quad (19)$$

The moment at the same point is

$$M_Z = M_0 + M_0' + Z\,(V_0 + V_0') \quad (20)$$

Earthquake Forces in the West Bay Superstructure

A general consideration of the action of the suspension bridge (Fig. 1(*a*)) is a necessary preface to the stress analysis. The San Francisco Anchorage is a massive concrete block and may be considered an upward extension of the rock. The rock mass of Yerba Buena Island forms the eastern anchorage, the cables being connected to grillages set in tunnels and securely held by the complete backfilling of the tunnels with concrete. The Central Anchorage is also massive but must be analyzed as a vertical cantilever 450 ft high. In the longitudinal direction, the towers are flexible columns and depend, to a large extent, on the cables acting as guys. In the transverse direction, they are comparatively stiff braced bents. The cables must be considered as transmitting heavy forces but as having small mass. Compared with their dimensions, the stiffening trusses are very flexible. At the towers and anchorages they are held in a transverse direction only. The floor system and the concrete

floor slabs have expansion joints at 120-ft intervals. For small movements, the friction at these joints is probably such that the floor system participates in chord deformations. With heavy chord stresses and consequent larger deformations, the friction would be broken. In both horizontal and vertical directions, the trusses are partly fixed at the towers by the friction of the expansion joints.

An idea of the flexibility of the system may be gained by the movement under the loading conditions assumed for the design. The center of the longer spans has a calculated range of vertical motion of 25 ft; the top of Tower 2, a horizontal motion of 6.5 ft. With a wind of 90 miles per hr, the calculated horizontal deflections of the center spans are 9.5 ft. The stresses, especially in the suspended structure, are controlled by these distortions. Compared with the deflections previously given, the amplitudes of any earthquake motion are very small and suggest that the resulting earthquake stresses would be rather small as compared to the other forces.

Earthquake Forces Transmitted to Anchorages through Cables.—As the anchorages are accelerated in a longitudinal direction, they, in turn, accelerate the attached cables and cause accelerations in the various elements suspended from the cables. The problem is to find the additional forces set up in the cables by reason of these accelerations. Because of the complexity of the suspension structure and the different periods of each of its masses, it was idealized as shown in Fig. 4(*b*). The earthquake effect was obtained by increasing the cable pull F by the coefficient $\frac{a}{g} = \alpha = 0.10$ (see Fig. 4(*a*)).

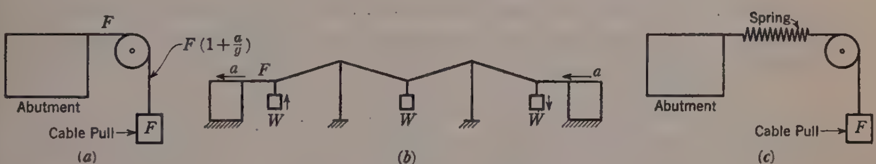


FIG. 4.—LONGITUDINAL CABLE PULL

As a second approach, the weight of the various spans may be replaced by equivalent weights at the centers of the three spans (Fig. 4(*b*)) and the increment of cable stress calculated when the anchorage has an acceleration of $\alpha\, g$, no allowance being made for the elongation of the cable. A calculation on this basis results in the change of stress in the cable of $0.7\, \alpha\, F$.

It is evident that the entire suspension system has many of the characteristics of a spring. The natural period (unloaded bridge) has been found by measurement to be 6.2 sec. The corresponding period for the fully loaded bridge would be 7.3 sec. The spring constant, or the additional cable stress for a 1-in. movement of the anchorage, is 35,000 lb for the two cables. Substituting in the spring equation, it is found that as far as dynamic action on the cables is concerned, the conditions may be replaced by the construction shown by Fig. 4(*c*), with the maximum earthquake force on the anchorage of $0.0265\, F$. It may be noted that, in any event, the different periods of the center and side span stiffening trusses would prevent resonance. It may be noted that in-

creasing F by αF would have no material effect on either of the end anchorages. The center anchorage was designed on the basis (at high unit stresses) of one of the twin bridges being completely removed.

Earthquake Stresses in Towers.—In the transverse direction, the towers are subjected, during an earthquake, to stresses arising from the inertia of the suspended structure, of the cables, and of the towers themselves. The suspended spans are free to move longitudinally so that, except for the friction of the roadway expansion joints and of the suspended span bearings, there are no longitudinal forces at the floor level.

The natural periods of transverse vibration of the stiffening trusses were calculated for various assumptions of loading and end conditions as shown in Table 1. For pinned ends, the fundamental period is

$$T_1 = \frac{2\,L^2}{\pi}\left(\frac{w}{E\,I\,g}\right)^{0.5} \qquad (21)$$

in which L = span length of stiffening truss, w = weight per foot of bridge, E = modulus of elasticity, I = moment of inertia of stiffening truss, and g = acceleration of gravity. For example, for assumption 2, $T_1 = 3.35 \times 10^6 \times 0.000\,000\,464 = 15.5$ sec. The first two harmonics are: $T_2 = \frac{T_1}{4} = 3.9$ sec; and, $T_3 = \frac{T_1}{9} = 1.7$ sec.

The periods in Table 1 are calculated on the basis that the suspenders are flexible and have no effect on the transverse vibrations. This is true only for small amplitudes; should the amplitudes build up to the amount necessary to cause large reactions at the towers, the suspenders and cables will act as flexible supports in the transverse direction, and change the period.

TABLE 1.—Computed Periods of Vibration

Assumption	Loading condition	End condition	Floor participation	Period, in Seconds	
				Center span	Side spans
1	Loaded	Pinned	No	18.5	4.9
2	Unloaded	Pinned	No	15.5	4.1
3	Unloaded	Pinned	Yes	14.0	3.7
4	Unloaded	Fixed	No	6.8	1.8
5	Unloaded	Fixed	Yes	6.1	1.6

The periods of the center spans were measured under unloaded conditions and found to be approximately 9.0 sec. The corresponding amplitudes were very small, a maximum of 0.2 in. Probably, under these conditions, there would be no movements of the intermediate expansion joints, and the friction at the towers would be sufficient to approach the fixed-end condition. With amplitudes sufficient to cause considerable chord stresses, the movement of the floor expansion joints would reduce the floor participation, the friction at the towers would not be sufficient to maintain the fixed-end condition, and the periods would approach those of assumptions 1 or 2, Table 1, depending upon the loading on the bridge.

The end reactions for condition 1, Table 1, for various periods of forced vibration are shown by Fig. 5, which shows that the fundamental periods of both center and side spans are considerably greater than those at which earthquake damage may be anticipated. The values of the reaction used in the design and, also (for purpose of comparison), the wind reaction, are shown in Fig. 6.

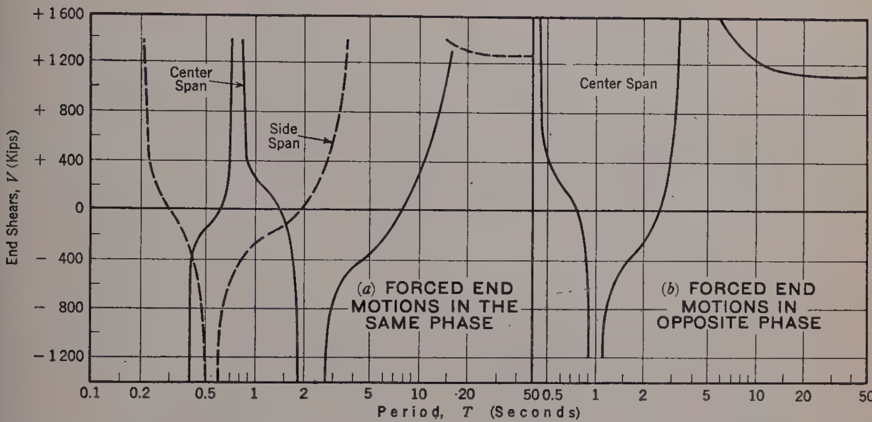


FIG. 5.—END SHEARS RESULTING FROM THE INERTIA OF A SUSPENDED STRUCTURE SUBJECTED TO FORCED VIBRATIONS AT ITS END SUPPORTS, WITH AMPLITUDES AND PERIODS CORRESPONDING TO 10% OF GRAVITY

The Appendix, under "Transverse Vibrations," shows the derivation of equations for moments and shears in the stiffening trusses. Completing the differentiation of Eq. 39*b* in the Appendix

$$V = \frac{d\,M}{E\,I} = E\,I\,\frac{d^3y}{dx^3} \quad\quad (22)$$

the equation for end shear is

$$V = E\,I\,m^3\,\frac{u_0}{2}\left[\left(\frac{1 - \cos m\,L}{\sin m\,L}\right) - \left(\frac{1 - \cosh m\,L}{\sinh m\,L}\right)\right] \quad\quad (23)$$

in which u_0 = amplitude of forced vibration. For example: When the forced period $T = 2.5$ sec, $L = 2{,}294.25$ ft; and $m = 3.41\ (10)^{-3}$; then, $m\,L = 7.823$ and $u_0 = 0.509$ ft for $\alpha = 0.10$. The end shear obtained by Eq. 23 would be 421,000 lb.

The cables are flexible. Until resonance is partly established it appears that, if a horizontal motion were imparted to them through the towers, the only inertia effect would be that imparted to a relatively small length of cable near the towers. Considering resonance and assuming small amplitudes, the theory of a tight string was applied, the actual tension in the cable and only the weight of the cable itself being considered. Under this assumption, the calculated natural fundamental periods for the bridge, loaded, are 5.9 and 3.1 sec for the center and side spans, respectively. It is also possible for the cable to swing as a compound pendulum; but the period, in the order of 15 sec for

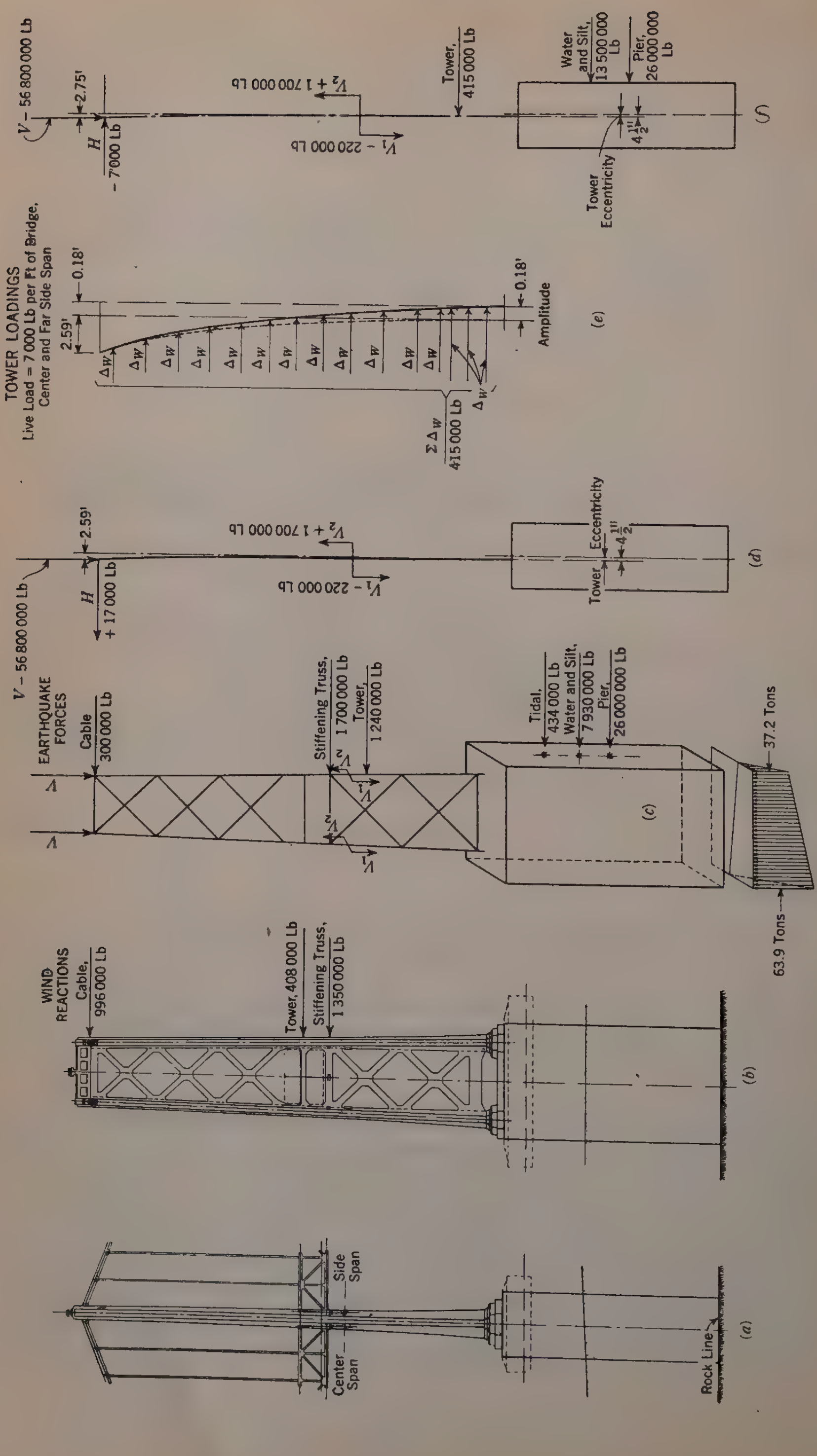


FIG. 6.—EARTHQUAKE FORCES ON TOWER AND PIER W-3

the center span, is entirely outside the range of damaging earthquake periods. Should the amplitudes reach any considerable amounts, the lateral resistance and mass of the suspended structure come into play, causing a change of periods.

To determine the properties of the main span cable considered as a compound pendulum, the following equations were applied: For center of gravity, assuming the cable curve to be a catenary, $y = \frac{f}{2}(e^{x/f} + e^{-x/f})$,

$$s\,h_g = \frac{f^2}{2}\sinh\frac{L}{f} + \frac{f\,L}{2} - 2\,f\,d\sinh\frac{L}{2\,f} \qquad (24)$$

in which s = length of cable between supports, and the other quantities are

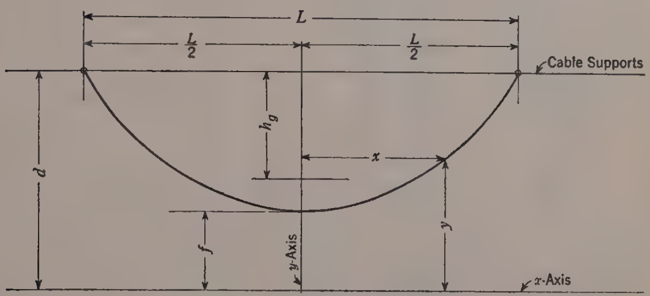


FIG. 7

as indicated by Fig. 7. For radius of gyration, r, about the axis through the cable supports,

$$s\,r^2 = \frac{f^3}{2}\left[\frac{1}{3}\sinh\frac{3\,L}{2\,f} + 3\sinh\frac{L}{2\,f}\right] - f^2\,d\sinh\frac{L}{f} - L\,f\,d + 2\,f\,d^2\sinh\frac{L}{2\,f} \qquad (25)$$

For this particular structure h_g = 150.3 ft and the period $T = 2\,\pi\left(\frac{C}{g}\right)^{0.5}$ = 15.0 sec in which $C = \frac{r^2}{h_g}$. This period checked the measured value as obtained by the seismometers.

Assuming resonance, the transverse reactions of the cables for forced vibrations of different periods are plotted on Fig. 8. From these curves, a horizontal reaction of 300 kips was chosen. Equations for the computation of these forces are shown in the Appendix under the heading "Flexible Vibrating String."

To the stresses in the tower resulting from the suspended span and cable reactions, those from transverse forces equal to 10% of the weight of the tower were added, giving the total tower stresses from an earthquake in the transverse direction.

In a longitudinal direction, except for a negligible amount of friction at the roadway level, the only additional forces acting on the tower during an earthquake are those arising from the inertia of the tower itself and such changes as there may be in the unbalanced horizontal component of the cables. Fig. 6(f) shows the deflected position of the tower under the most severe

assumption of live load and temperature. The tower tops are held in position by the cables acting as guys. Compared to the other forces, the force required to deflect the towers is very small.

In a longitudinal earthquake, one may assume that the base of the tower is moved and that the top of the tower remains practically fixed in space. The

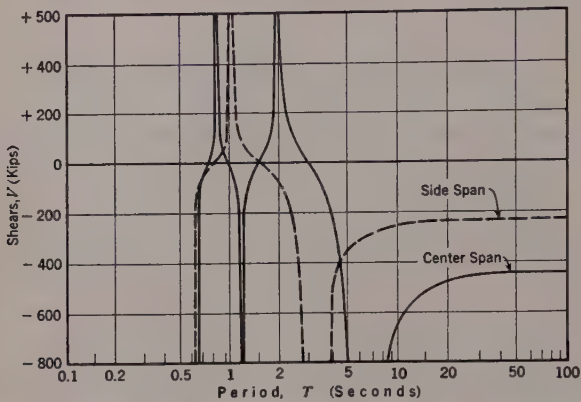


FIG. 8.—TRANSVERSE CABLE REACTIONS RESULTING FROM INERTIA OF A CABLE SUBJECTED TO FORCED VIBRATIONS AT THE TOWER TOPS, WITH AMPLITUDE AND PERIODS CORRESPONDING TO 10% OF GRAVITY

earthquake forces on the various elements of the tower (see Fig. 6(*e*)) are assumed as proportional to their respective amplitudes.

The value of the displacement Δ and all forces except H can be determined by the "Modified Dana Method"[4] and H can be found by the following equations: For a horizontal force at the end of a cantilever, the equation for the deflection is:

$$\Delta_H = \sum_0^L \frac{M\,y\,dy}{E\,I} = \sum_0^L \frac{H\,y^2\,dy}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (26)$$

for a vertical force:

$$\Delta_V = \sum_0^L \frac{M\,y\,dy}{E\,I} = \sum_0^L \frac{V\,(\Delta - \delta_y)\,y\,dy}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots (27)$$

and, for a moment:

$$\Delta_M = \sum_e^L \frac{M\,dy}{E\,I} = \sum_e^L \frac{M\,(y - e)\,dy}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots (28)$$

In Eqs. 26, 27, and 28: Δ = horizontal displacement of top of tower; y = vertical distance from top of tower; δ_y = horizontal displacement at distance y from top of tower; V = vertical load; M = moment; and e = vertical distance from top of tower to a point where moment load is applied. From these three fundamental equations the various forces due to wind, loading, earthquake,

[4] "Tests on Structural Models of Proposed San Francisco-Oakland Suspension Bridge," by George E. Beggs, Raymond E. Davis, M. Am. Soc. C. E., and Harmer E. Davis, Assoc. M. Am. Soc. C. E., *Publications in Engineering*, Univ. of California, Vol. 3, p. 151.

etc., were obtained. The displacements for this particular structure are shown in Fig. 6. Each value of Δ_W in Fig. 6(*e*) was assumed proportional to its respective amplitude.

East Bay Crossing

The principal structures in the East Bay Crossing consist of the cantilever structure and the five 508-ft spans which have a common junction at Pier E-4, each system being anchored in a longitudinal direction at Pier E-1 and Pier E-9, respectively. Consideration may be given to the conditions before resonance is established, stresses during resonance, and the factors inherent in the construction to destroy resonance.

The natural longitudinal period of the unloaded cantilever structure has been determined by measurement as approximately 1.5 sec. With the bridge fully loaded, this period would be increased to about 1.8 sec. This has been interpreted as indicating that if an impulse were applied at Pier E-1, it would travel across the span and reach Pier E-4 in 0.9 sec. Therefore, with a period 1.8 sec, and with a condition of no acceleration at either end, the maximum acceleration would occur at the center of the span. The assumption has been made, therefore, that for the west anchor arm, an average acceleration of $\frac{2}{\pi}$ times 10% could be used instead of 10% on the entire structure. The resulting forces are assumed to be statically applied. By this assumption, the maximum horizontal force applied to Pier E-1 from the inertia forces of the cantilever structure, including live load, is 5,000,000 lb. The similar assumptions for the longitudinal stresses in the 508-ft spans are shown in Fig. 6(*d*).

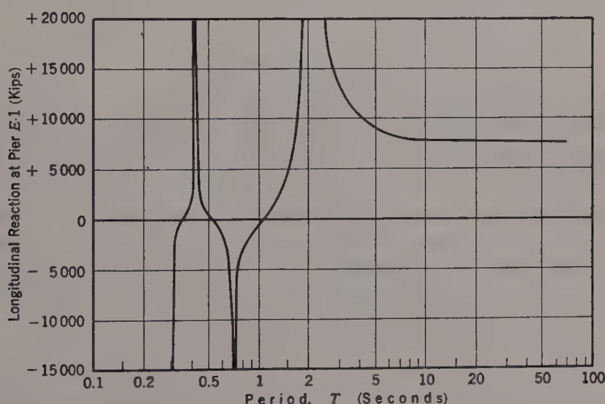


Fig. 9.—Reactions at Pier E-1 Resulting from the Inertia of the Cantilever Structure Subjected to Forced Longitudinal Vibrations at That Pier, with Amplitudes and Periods Corresponding to 10% of Gravity

The horizontal forces from the cantilever, acting on Pier E-1 in the case of continued forced harmonic vibrations with no allowance for damping, are shown in Fig. 9. Equations for the calculation of these forces are shown in the Appendix, under the heading "Longitudinal Vibrations."

In the transverse direction, the design was made on the basis of a horizontal static force of 10% gravity. With the elasticity of the structures, especially the cantilever, some reduction could have been safely made. The measured natural period of this structure (unloaded) is 3.44 sec. Under load, it would be increased to about 3.9 sec.

The following are among the reasons that make resonance improbable:

(1) The natural periods of the structure (fundamental mode) are considerably in excess of those of the probable ground motions; and

(2) The floor expansion joints will have an action and effect similar to the foregoing in the case of the suspension structure.

Effect of Earthquake Stresses on Design.—Although the earthquake stresses in the various parts of the bridge when calculated in accordance with the foregoing assumptions are of considerable magnitude, they had a comparatively small effect on the proportions of the bridge. Generally, the maximum foundation pressures occur when seismic forces are considered. On the other hand, the piers are no larger than would ordinarily be used considering the size of the superstructure. The ratio of the exposed area to the mass of the superstructure is such that the horizontal forces from wind are of the same order as those from the earthquake assumptions. For these reasons, the increased cost arising from provision for a 10% seismic coefficient was very small, probably less than 5%.

Vibration Periods of Bridge.—In the foregoing, reference has been made to the measured vibration periods of the bridge. These measurements were made by engineers of the U. S. Coast and Geodetic Survey and the results published by the Seismological Society of America.[5]

ACKNOWLEDGMENTS

The project was designed and its construction supervised by the San Francisco-Oakland Bay Bridge Division of the Department of Public Works acting under the direction of the California Toll Bridge Authority. C. H. Purcell, Assoc. M. Am. Soc. C. E., and Charles E. Andrew and Glenn B. Woodruff, Members, Am. Soc. C. E., served respectively as chief engineer, bridge engineer, and engineer of design. The writers wish to thank Franklin P. Ulrich, M. Am. Soc. C. E., Philip N. Fletcher, Assoc. M. Am. Soc. C. E., and Dean S. Carder for their assistance in assembling data for the preparation of this paper.

APPENDIX

THEORY

The derivations of some of the formulas that have been used in calculating the effects of forced harmonic vibrations on certain parts of the bridge are

[5] "Observed Vibration of Bridges," by Dean S. Carder, *Bulletin of the Seismological Society of America*, Vol. 27, No. 4, October, 1937.

given herein. In these derivations, ideal cases only have been considered; that is, the bridge structure has been assumed to be of such dimensions that it can be considered as having the characteristics of a thin rod, and the cable (for transverse vibrations) has been assumed as a flexible vibrating string. Transverse distortions in longitudinal vibrations, variations in areas and moments of inertia, some of the effects of the addition of inert masses to the bars, and similar other factors that will affect numerical values, have not been considered. The effect of damping has been omitted entirely. For these reasons, the numerical results obtained for any particular period of forced vibration may be considerably in error. However, if calculations are made for a series of different periods, the general characteristics of the induced stresses thus determined can be used as one of the guides in the selection of design stresses.

Transverse Vibrations

The differential equation for the transverse vibrations of thin uniform rods is:

$$\frac{\partial^4 y}{\partial x^4} + \frac{w_r}{E\,I\,g}\frac{\partial^2 y}{\partial t^2} = 0 \qquad (29)$$

in which: w_r = weight per unit length of rod; E = modulus of elasticity; I = moment of inertia of the rod; and t = time. To solve this equation, assume $y = u \cos p\,t$ in which amplitude u is a function of x only. Then

$$\frac{\partial^2 y}{\partial t^2} = -u\,p^2 \cos p\,t \qquad (30a)$$

and

$$\frac{\partial^4 y}{\partial x^4} = \frac{\partial^4 u}{\partial x^4} \cos p\,t \qquad (30b)$$

Substituting these values in Eq. 29,

$$\frac{d^4 u}{dx^4} - \frac{p^2\,w_r\,u}{E\,I\,g} = 0 \qquad (31a)$$

Let $\frac{p^2\,w_r}{E\,I\,g} = m^4$; then

$$\frac{d^4 u}{dx^4} - m^4\,u = 0 \qquad (31b)$$

A solution of Eq. 31*b* is:

$$u = A \cos m\,x + B \sin m\,x + H \cosh m\,x + K \sinh m\,x \qquad (32)$$

Natural Period of Vibration of a Rod Pinned at Both Ends.—The end conditions are: At $x = 0$ and at $x = L$, $y = 0$ (no deflection), and $\frac{\partial^2 y}{\partial x^2} = 0$ (moment = 0). Therefore, $u = 0$ and $\frac{d^2 u}{dx^2} = 0$, when $x = 0$ and $x = L$.

Applying the foregoing conditions at $x = 0$ gives $A = H = 0$. Applying the same conditions at $x = L$ gives:

$$0 = A \cos m L + B \sin m L + H \cosh m L + K \sinh m L \quad \text{(33a)}$$

and

$$0 = - A \cos m L - B \sin m L + H \cosh m L + K \sinh m L \quad \text{(33b)}$$

Since $A = H = 0$, the addition and subtraction of these two equations give, respectively,

$$2 K \sinh m L = 0 \quad \text{(34a)}$$

and

$$2 B \sin m L = 0 \quad \text{(34b)}$$

Since $\sinh m L$ cannot be zero, K must be zero. If $B = 0$, y is always zero and the rod is at rest. Therefore $\sin m L$ must be zero. This gives: $m L = \pi$ or 2π or 3π etc. $\cdots$ or $n \pi$; $m = \frac{\pi}{L}$; and $m^4 = \frac{\pi^4}{L^4} = \frac{p^2 w_r}{E I g}$. Therefore, for the fundamental mode of vibration,

$$p = \frac{\pi^2}{L^2 \left(\frac{w_r}{E I g}\right)^{0.5}} \quad \text{(35)}$$

For higher modes when $m = \frac{n \pi}{L}$,

$$p_n = \frac{n^2 \pi^2}{L^2 \left(\frac{w_r}{E I g}\right)^{0.5}} \quad \text{(36)}$$

The natural period of vibration is

$$T = \frac{2 \pi}{p_n} = \frac{2 L^2}{n^2 \pi} \left(\frac{w_r}{E I g}\right)^{0.5} \quad \text{(37)}$$

Rod Subjected to Simple Harmonic Motion in a Transverse Direction at Each End.—Assume harmonic motions of both ends to be in phase. The end conditions are: At $x = 0$ and at $x = L$, $y = u_0 \cos p t$, and $\frac{\partial^2 y}{\partial x^2} = 0$. Therefore, $u = u_0$ and $\frac{d^2 u}{dx^2} = 0$, when $x = 0$ and $x = L$.

From these end conditions it is found that

$$A = H = \frac{u_0}{2} \quad \text{(38a)}$$

$$K = \frac{u_0}{2} \frac{(1 - \cosh m L)}{\sin m L} \quad \text{(38b)}$$

and

$$B = \frac{u_0}{2} \frac{(1 - \cos m L)}{\sin m L} \quad \text{(38c)}$$

When the constants in Eqs. 38 are known, moments and shears can be calculated by means of the following relationships:

$$M = E\,I\,\frac{d^2y}{dx^2} \qquad (39a)$$

and

$$V = \frac{dM}{dx} = E\,I\,\frac{d^3y}{dx^3} \qquad (39b)$$

Rod Subjected to Simple Harmonic Motion in a Transverse Direction at Each End.—Assume harmonic motions at the ends to be 180° out of phase with each other. Then the end conditions will be (when $x = 0$, $y = u_0 \cos p\,t$, or $u = u_0$) $\frac{\partial^2 y}{\partial x^2} = 0$, or $\frac{d^2u}{dx^2} = 0$. When $x = L$, $y = u_0 \cos (p\,t + \pi)$ or $u = -\,u_0$, and $\frac{\partial^2 y}{\partial x^2} = 0$, or $\frac{d^2u}{dx^2} = 0$.

From these end conditions it is found that:

$$A = H = \frac{u_0}{2} \qquad (40a)$$

$$K = -\frac{u_0}{2}\left(\frac{1 + \cosh m\,L}{\sinh m\,L}\right) \qquad (40b)$$

and

$$B = -\frac{u_0}{2}\left(\frac{1 + \cos m\,L}{\sin m\,L}\right) \qquad (40c)$$

Longitudinal Vibrations

Let u = longitudinal displacement at any point along the rod; x = distance along the rod; $\frac{w_r}{g}$ = mass of rod per unit volume; and E = modulus of elasticity. Then $E\,\frac{\partial^2 u}{\partial x^2}\,dx = \frac{w_r}{g}\,dx\,\frac{\partial^2 u}{\partial t^2}$; or

$$\frac{\partial^2 u}{\partial t^2} = c^2\,\frac{\partial^2 u}{\partial x^2} \qquad (41)$$

in which

$$c^2 = \frac{g\,E}{w_r} \qquad (42)$$

Assume $u = \xi \sin p\,t$ where ξ is a function of x only and p is a constant. Then

$$\frac{\partial^2 u}{\partial t^2} = -\,p^2\,\xi \sin p\,t \qquad (43a)$$

and

$$\frac{\partial^2 u}{\partial x^2} = \frac{d^2\xi}{dx^2}\sin p\,t \qquad (43b)$$

Therefore, $-p^2 \xi \sin p t = c^2 \frac{d^2 \xi}{dx^2} \sin p t$; or

$$\frac{d^2 \xi}{dx^2} = -\frac{p^2}{c^2} \xi \qquad (44)$$

and

$$\xi = C_1 e^q + C_2 e^{-q} \qquad (45a)$$

$$\xi = C_1 \cos \frac{p x}{c} + C_1 i \sin \frac{p x}{c} + C_2 \cos \frac{p x}{c} - C_2 i \sin \frac{p x}{c} \qquad (45b)$$

$$\xi = A \cos \frac{p x}{c} + B \sin \frac{p x}{c} \qquad (45c)$$

in which, to simplify typography, $q = \frac{i p x}{c}$.

Natural Periods of Vibration for Rod Fixed at One End and Free at the Other.—The end conditions are: Where $x = 0$, $u = 0 = \xi$, and where $x = L$, $\frac{\partial u}{\partial x} = 0 = \frac{d\xi}{dx}$. Furthermore, $0 = A$ (from $x = 0$ and $u = 0$) $= \frac{B p}{c} \cos \frac{p L}{c}$ $\left(\text{from } x = L \text{ and } \frac{\partial u}{\partial x} = 0\right)$.

If $B = 0$, there is no motion; therefore $\cos \frac{p L}{c} = 0$ and $\frac{p L}{c} = \frac{\pi}{2}$, or $\frac{3 \pi}{2}$, or $\frac{5 \pi}{2}$, etc. $= \frac{(2 n - 1)}{2} \pi$, in which n is an integer. Also,

$$p = \frac{(2 n - 1) \pi c}{2 L} \qquad (46)$$

The natural period $T_n = \frac{2 \pi}{p}$; and, for the fundamental period,

$$T_1 = 2 \pi \times \frac{2 L}{\pi c} = \frac{4 L}{c} \qquad (47)$$

Rod Subjected to Harmonic Motion in a Longitudinal Direction at One End (Other End Free).—Let u_0 = half amplitude of forced harmonic motion at $x = 0$ (that is, u_0 is the maximum displacement). The end conditions are: When $x = 0$, $\xi = u_0$ and $A = u_0$. At the free end ($x = L$), there is no stress. Therefore, $\frac{d\xi}{dx_{(x=L)}} = 0$, so that

$$\frac{d\xi}{dx} = -\frac{A p}{c} \sin \frac{p x}{c} + \frac{B p}{c} \cos \frac{p x}{c} \qquad (48a)$$

$$0 = \frac{-u_0 p}{c} \sin \frac{p L}{c} + \frac{B p}{c} \cos \frac{p L}{c} \qquad (48b)$$

and

$$B = \frac{u_0 \sin \frac{p L}{c}}{\cos \frac{p L}{c}} = u_0 \tan \frac{p L}{c} \qquad (48c)$$

Then

$$u = u_0 \left(\cos \frac{p\,x}{c} + \tan \frac{p\,L}{c} \sin \frac{p\,x}{c} \right) \sin p\,t \quad \text{(49a)}$$

and

$$\frac{\partial u}{\partial x} = \frac{u_0\,p}{c} \left(- \sin \frac{p\,x}{c} + \tan \frac{p\,L}{c} \cos \frac{p\,x}{c} \right) \sin p\,t \quad \text{(49b)}$$

The total longitudinal stress $= A\,E \frac{\partial u}{\partial x}$, in which A = area. At $x = 0$, the maximum longitudinal stress equals

$$\frac{A\,E\,u_0\,p}{c} \tan \frac{p\,L}{c}.$$

Flexible Vibrating String

Fig. 10 shows an elementary length of a vibrating string. Let T = tension in string; L = length (distance between supports); and ρ = mass per unit length of string. The forces acting on length ds in the y-direction are: $\rho\,ds \frac{\partial^2 y}{\partial t^2}$ due to acceleration, and $T \frac{\partial^2 y}{\partial x^2} dx$ due to change in direction of T in the distance dx. Since the angles are small, ds can be assumed to be equal to dx and $\rho\,dx \frac{\partial^2 y}{\partial t^2}$ can be substituted for $\rho\,ds \frac{\partial^2 y}{\partial t^2}$. Then

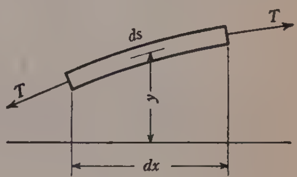


Fig. 10

$$\rho\,dx \frac{\partial^2 y}{\partial t^2} = T \frac{\partial^2 y}{\partial x^2} dx \quad \text{(50)}$$

or,

$$\frac{\partial^2 y}{\partial t^2} = c^2 \frac{\partial^2 y}{\partial x^2} \quad \text{(51)}$$

in which $c^2 = \frac{T}{\rho}$. A solution of Eq. 51 is

$$y = \left(A \cos \frac{p\,x}{c} + B \sin \frac{p\,x}{c} \right) \sin p\,t \quad \text{(52)}$$

Natural Periods of Vibration of a String Under Tension.—The end conditions are: When $x = 0$, $y = 0$; therefore, $A = 0$. When $x = L$, $y = 0$; therefore, $B \sin \frac{p\,L}{c} = 0$.

If $B = 0$, there is no motion; therefore, $\sin \frac{p\,L}{c}$ must equal zero, and $\frac{p\,L}{c} = \pi$ or $2\,\pi$ or $3\,\pi$, etc.; and $p = \frac{n\,\pi\,c}{L}$ (in which n is an integer); natural period $= \frac{2\,\pi}{p} = \frac{2\,L}{n\,c}$; and fundamental period $T_1 = \frac{2\,L}{c}$.

Flexible String Under Tension and Subjected to Simple Harmonic Motion in a Transverse Direction at Each End.—Let u_0 = the half amplitude of forced harmonic motion at each end of a string. The end conditions are: When $x = 0$, $y = u_0 \sin p\,t$; therefore, $A = u_0$. When $x = L$, $y = u_0 \sin p\,t$; and $u_0 = u_0 \cos \frac{p\,L}{c} + B \sin \frac{p\,L}{c}$. Therefore,

$$B = \frac{u_0 \left(1 - \cos \frac{p\,L}{c}\right)}{\sin \frac{p\,L}{c}} \qquad (53)$$

Then

$$y = u_0 \left[\cos \frac{p\,x}{c} + \frac{1 - \cos \frac{p\,L}{c}}{\sin \frac{p\,L}{c}} \sin \frac{p\,x}{c} \right] \sin p\,t \qquad (54a)$$

and

$$\frac{\partial y}{\partial x} = \frac{u_0\,p}{c} \left[- \sin \frac{p\,x}{c} + \frac{1 - \cos \frac{p\,L}{c}}{\sin \frac{p\,L}{c}} \cos \frac{p\,x}{c} \right] \sin p\,t \qquad (54b)$$

The transverse component of tension $= T \frac{\partial y}{\partial x}$; when $x = 0$,

$$\frac{\partial y}{\partial x} = \frac{u_0\,p}{c} \frac{\left(1 - \cos \frac{p\,L}{c}\right)}{\sin \frac{p\,L}{c}};$$

and, when $x = L$,

$$\frac{\partial y}{\partial x} = \frac{-u_0\,p}{c} \frac{\left(1 - \cos \frac{p\,L}{c}\right)}{\sin \frac{p\,L}{c}}.$$

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

TRANSATLANTIC SEAPLANE BASE, BALTIMORE, MARYLAND

By W. Watters Pagon,[1] M. Am. Soc. C. E.

Synopsis

Two major topics are described in this paper: (*a*) The difficult foundation conditions; and (*b*) the unusual hangar design dictated by these conditions, caused by the character of the mud fill which was originally placed in the Baltimore Municipal Airport, in Baltimore, Md. The paper deals only with the seaplane base, the first portion of which comprises a hangar 270 ft by 190 ft with 35 ft clear height, built entirely of steel, and a brick office building in the form of a half octagon, 66 ft by 125 ft, both of which are leased to the Pan American Airways System.

Brief History

First Stage.—In 1928 the voters of Baltimore approved the expenditure of $1,500,000 for the construction of an airport, which must be a combination landplane and seaplane base. It represented a recrudescence of the spirit which produced the famous Baltimore Clippers of one hundred years ago. In 1930 an additional $2,500,000 was voted. The site selected for the airport, although ideal from the operating standpoint and unexcelled elsewhere (see Fig. 1), was exceedingly poor from the engineering and financial standpoint. Of the 360 acres in the site, only 70 were fast land, and the remainder was open water having depths ranging from 6 ft to 18 ft. There were also 71.5 acres just outside of the field for use as automobile parking lots. The bed of the Patapsco River here consists of two, or more, marsh deposits, which are indicated by fossil leaves to be of Pleistocene (Talbot) age, and which extend nearly to 60 ft below mean low tide.

The site is on the north shore of the Patapsco River, about 5 miles in a direct line, or 6 miles by automobile, and 15 min from the Civic Center. It is well served by highways, and is bounded on one side by a wide highway. A steam railroad freight line and a street-car line, which also serve the steel mills at Sparrows Point, Md., are nearby. Beyond the car line is the village of

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **February 15, 1941.**

[1] Cons. Engr., Baltimore, Md.

Dundalk, Md. The other three sides are bounded by open water, except for a portion of the south side, where there is abutting industrial property. A considerable portion of the area of the field is beyond the pierhead line, by special federal permission.

As shown in Fig. 1, the Patapsco River is an estuary which opens into Chesapeake Bay and provides a minimum seaplane water runway 2 miles long directly in front of the airport, in a northeast-southwest direction, with almost unlimited length in other directions.

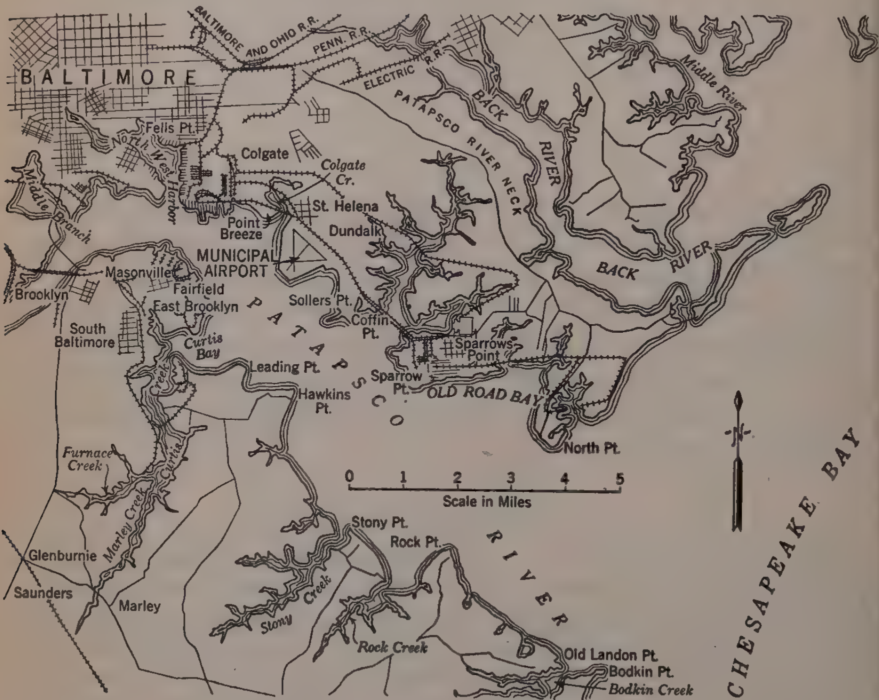


FIG. 1.—LOCATION OF AIRPORT

Along two sides of the airport permanent bulkheads, called *A*, *B*, and *C*, were constructed, the third side being bounded merely by a "mud fence," called *D* (see Fig. 2). Before construction of bulkhead *B* the river bed was dredged to 35 ft. The type chosen is an apron 38 ft wide, supported on plumb piles and two rows of batter piles, and surmounted by a concrete face wall with its top at 7.0 ft above mean low tide (see Fig. 3). A riprap toe was planned, but not placed.

Data for the wind rose in Fig. 2 were supplied by the U. S. Weather Bureau. The prevailing winds are southwesterly, the highest average velocity is northwesterly, and there is no perceptible wind 1% of the time. The normal tide range is 13 in.; high tides rise at times (due to wind) to +5.0 ft; in the tropical hurricane of October, 1933, the maximum height reached was +8.0 ft. Hence the hangar floor is at El. +8.0 and the office building floor at +9.0.

Material being excavated from a nearby ship anchorage was first planned for use in the airport, at a considerable saving, but the material that was actually received proved to be of very poor character and yielded disappointing results, as described subsequently. Other mud came from points in Baltimore Harbor, of which some was so soft that a 30-ft pole could be pushed entirely into it by one man. At first the material was placed with bottom-dump scows; later it was dropped into the river outside of the airport and rehandled by a suction dredge. The result was that, when it arrived in place, the fill consisted mostly of semiliquid mud. Such coarse material as it contained settled at the end of the discharge line, which was close to bulkhead *D*, and the fines found their way to the area to be occupied by the hangar described in this paper.

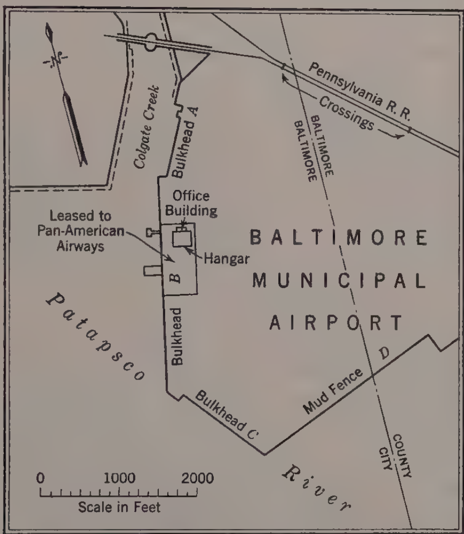


FIG. 2.—GENERAL LAYOUT OF THE AIRPORT

Long before completion of the fill the bulkheads showed distress. Bulkhead *B* was first reinforced with "anchor piles"; later a sand berm was placed, inside and outside of the sheet piling, thus filling one trench again to a depth of 15 ft or 20 ft. This trench had been dredged previously to 35 ft before pile driving was commenced. Some of the mud also flowed into this trench before the sheeting was completed. Bulkhead *C* was only a line of steel sheet piles, anchored back to deadmen, but, due to corrosion and to the loss of several anchorages, it has been buried by constructing a beach in front, riprapped with old paving blocks.

Second Stage.—In 1931 the city administration sought engineering advice as to what to do with the project. By that time the site had been almost completely filled in. The project constituted a mud fill so liquid that it had overflowed bulkhead *B* on at least two occasions, and had then become quiescent at an angle of repose of approximately 6 ft in 2,600 ft, or about 0° 8′. The answer given by the consulting engineers was that there was nothing that

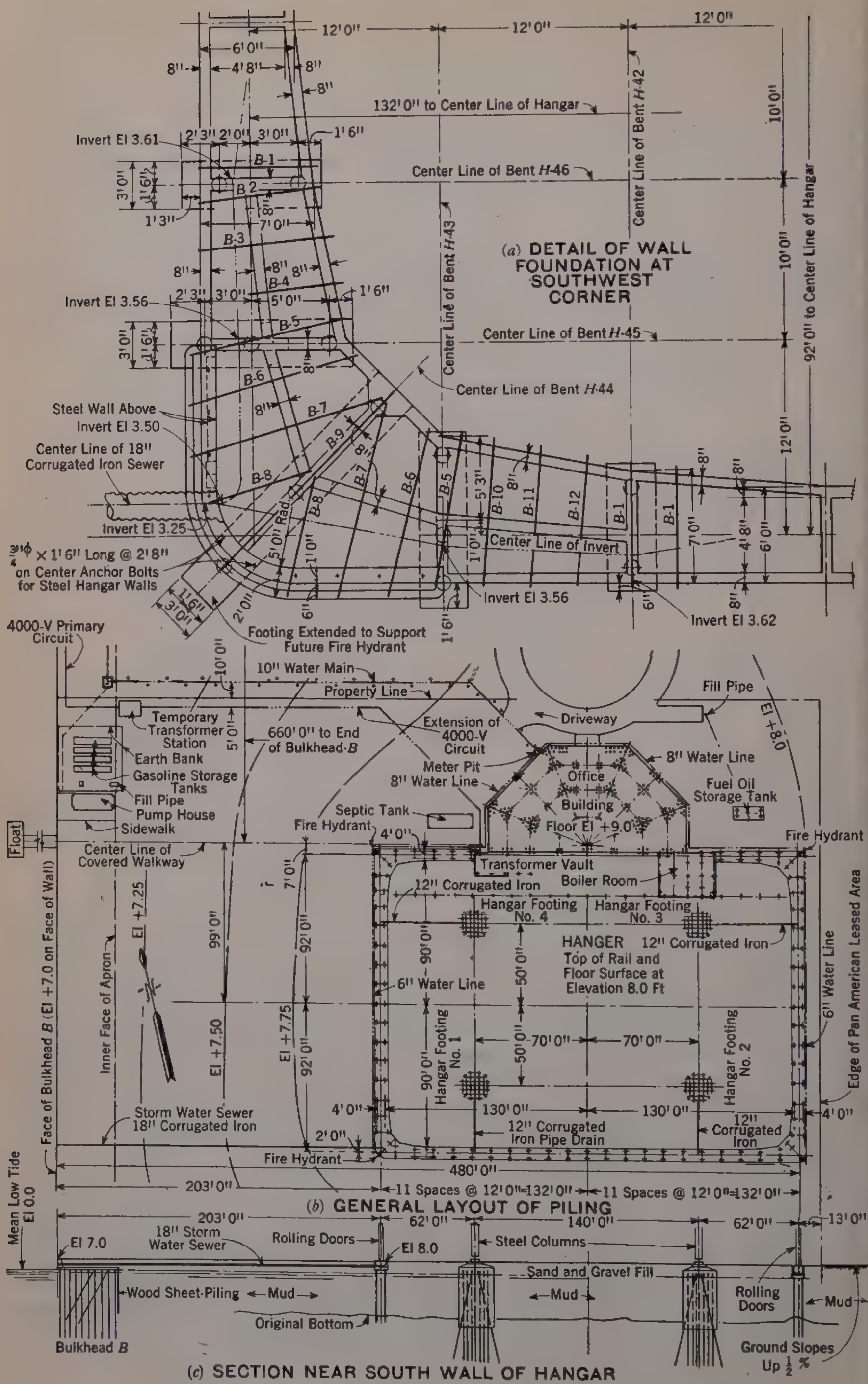


FIG. 3.—FOUNDATION PLAN OF SOUTH WALL OF HANGAR

could be done except to let the fill lie in the sun and dry out for a period of twelve to fifteen months, and then to place about 3 ft of porous, sand-clay material on top of the mud by pumping a granular fill from the river bed.

Third Stage.—Following this report no further work was done until 1936, when the city entered into an agreement with the Public Works Administration (PWA) for approximately $1,500,000, and into an agreement with Pan American Airways for the establishment of one of two alternate transatlantic seaplane bases, at Baltimore and at La Guardia Airport, New York, and an emergency base at Charleston, S. C. At this time the city had expended about $3,000,000 and had about $1,000,000 still available. The facilities to be described were constructed by the city and leased to the company.

A contract was let for the placing of granular fill upon the mud by two methods: (*a*) By dredging from the river; and (*b*) by hauling from a low sandy hill about 2 miles away. This hill was a deposit of white to pink fine sand, with 10% to 15% clay, of the Raritan Formation, capped with gravels of the Sunderland Terrace. The fill was to consist of approximately 90% sand and gravel, with a strict limitation on the fines. The work of placing this fill is not the concern of this paper. It is sufficient to state that the resulting depth of cover upon the mud varied according to the extent of the mud waves which were forced up. At the hangar site it varied from 5 ft to 17 ft. Just south of the hangar, where two mud waves met, there was no cover, but instead the mud wave stood some 5 ft above finished grade.

After commencement of the fill the first problem was the field layout. The city had previously leased a site for a seaplane factory, and the existence of this lease limited the layout of both the field and the base. The latter, naturally, was located along bulkhead *B*, and it was desired to place it as near the north end as possible, for ready access from the land. However, two conditions fixed its position: (*a*) It was felt to be unsafe to taxi seaplanes farther up Colgate Creek, between the airport and the loading wharf of the Western Electric Company, Inc., than to the point chosen; and (*b*) it was necessary to maintain a width of 1,000 ft for the north-south landplane runway, which was planned for instrument landings. This width of 1,000 ft was twice the original requirement, but according to the latest requirement it should have been 1,400 ft. The engineering effect of these limitations was to place the seaplane base at a point where the mud consistency was probably at its lowest value.

The Mud

The limitations of the site cannot be appreciated without an understanding of the mud underlying the site of the seaplane base. The depth from the hangar floor (El. +8.0) to the original river bottom (which was soft) was about 26 ft. There was a shallow channel across the site, at which the depth of mud was about 30 ft. The mud particles were so fine that the mass was colloidal. The particles that overlie the old marshes have been deposited for hundreds, perhaps thousands, of years in an open river, about 2 miles wide, subject to waves about 3 ft high, and in general at a distance of about 3 miles from the mouths of the tributary streams which had transported them. All of this is evidence of the fineness of the particles. The writer experimented with every

mechanical, chemical, and electrical means known and suggested by colloidal chemists, for reducing the liquidity of the mud, but found that only by wicks could the colloidal structure be destroyed. This idea formed part of the basic design concept.

The field slope was 1.0% (later 0.8%) upward from bulkhead *B*, for surface runoff. The runway is approximately 600 ft from the hangar, and is intended for instrument landings. The stability of the fill, therefore, was reduced by this surface slope. There was also the possibility of lateral displacement of the surface, due to jarring of landing airplanes, and the doubt, on the part of many of the public, as to the safety of bulkhead *B*, in which doubt, however, the writer did not concur. The first two reasons were sufficient for the foundation design that will be described. The cost of each main column foundation was nearly $20,000, and this would vary only a few per cent with the amount of column load; hence, there was introduced, also, a potent financial reason for the design.

When the work of filling, under "Third Stage," was begun, the mud fill had shrunk vertically from 3 to 5 ft, the exact amount being impossible to determine. Shortly after it had been pumped in, walking on its surface was impossible, because it could be stirred readily with a 6-ft stick. After eighteen months of drying in the sun there was a "crust" of indeterminate depth. If the depth to the bottom of the mud cracks is used, it was about 8 to 10 in.; but below the cracks was about 6 to 10 in. of putty-like mud. The topmost portion had dried to such an extent that it had reached its shrinkage limit, and then, on further loss of water, had turned from dark gray to a dirty white color. On the average the cracks had a top width of about 3 in., and occurred (in crude hexagonal pattern) on the average about every 10 in., indicating a linear contraction of perhaps 30%. The width of crack varied almost linearly down to the point where it closed, at which point the material was somewhere in the vicinity of the plastic limit.

Walking over this 18-month old cracked crust, with the underlying mud yielding under the weight at each step, created the sensation of walking on a tight bedspring. By jumping up and down at the proper frequency the entire area of crust, for a distance of 50 ft or more, could be set in oscillation. Under such forces the mud acted like an elastic jelly, and there was no evidence of permanent strain or set. When a chunk of the drier portion was lifted out, the piece would have a total depth of nearly 12 in., and the bottom break occurred where the consistency was about that of putty when it is dry enough not to adhere to the hands.

In 1931 the writer had had tests made by the U. S. Bureau of Public Roads on two mud samples and one sand sample from the berm under bulkhead *B*. The results were as given in Table 1, the particle size being graded as follows:

Grade	Size in millimeters (see Table 1)
Coarse sand	2 to 0.25
Fine sand	0.25 to 0.05
Silt	0.05 to 0.005
Clay	less than 0.005
Colloids	less than 0.001

Some unit weights of the mud were:

Moisture	Weight, in lb per cu ft
Dry	88
32% moisture	115
Plastic limit	110
Liquid limit	93

TABLE 1.—SOIL TESTS BY THE U. S. BUREAU OF PUBLIC ROADS

Sample No.	Particle Size, in Millimeters (Percentage of Moisture)							Type of material	Liquid limit	Plasticity index	Shrinkage		Moisture Equivalent		Specific gravity	Limit shrinkage	Volume change
	>2	2 to 0.25	0.25 to 0.05	0.05 to 0.005	<0.005	Total <2	<0.001				Limit	Ratio	Centrifugal	Field			
S-6265	0	1	7	23	69	100	32	mud	84	44	32	1.4	95[a]	60	2.64	11	39
S-6266	0	2	1	26	71	100	31	mud	85	43	31	1.4	91[a]	61	2.65	11	42
S-6264	46	68	12	11	9	100	7	sand	..	..	..	..	..	..	..	..	..

[a] Waterlogged, indicating a "muck" soil.

Below the plastic limit the material develops some resistance to pressure, the critical point which marks the change from a fluid to this state being generally at about the plastic limit. Since plastic limit equals liquid limit minus plasticity index, the plastic limit was 40% and 42%, respectively.

An interesting comparison is afforded by tests[2] at the National Bureau of Standards, in which it was found that a "Maryland Clay," when extruded from a $2\frac{9}{32}$-in. round cylinder, through a $\frac{1}{2}$-in., sharp-edged die, showed zero pressure at a water content of about 44% of the dry weight, and for less moisture showed a hyperbolic curve of pressure versus moisture. At 18% moisture the pressure reached 400 lb per sq in.

One sample, semiliquid, had been taken from within the area of the seaplane base; the other, from within the crust, was taken from about 1,000 ft south. The results of the compression versus load, and compression versus time, tests are given in Figs. 4 and 5. A similar sample placed in an open-mouth jar, with a protruding and overhanging wick, dried at office temperature in a few weeks to about one third its original cubic volume, whereas a companion sample, without wick, was little changed at the end of six months. Water dripped from the end of the wick. When the mud is at its driest condition its consistency is nearly that of slate, and it will take a high polish.

CONDITIONS AT THE SEAPLANE BASE SITE

When work on the seaplane hangar was begun, under the third stage of the work, the following facts and conditions obtained:

(*a*) The granular fill over the building area had a depth of 5 to 17 ft;

(*b*) There were unfounded fears on the part of many citizens that bulkhead *B* could not be regarded as safe;

[2] "Relation Between Moisture Content and Flow-Point Pressure of Plastic Clay," by Ray T. Stull and Paul V. Johnson, *Research Paper RP 1186*, *Journal of Research*, National Bureau of Standards, Vol. 22, January–June, 1939, p. 329.

(c) The process of placing the granular fill at this site had pushed a mud wave across the site, such that a 1-in. steel pipe, left over the week end from a wash boring operation, was found to be 15 ft (at the top) from its original position (see Fig. 6);

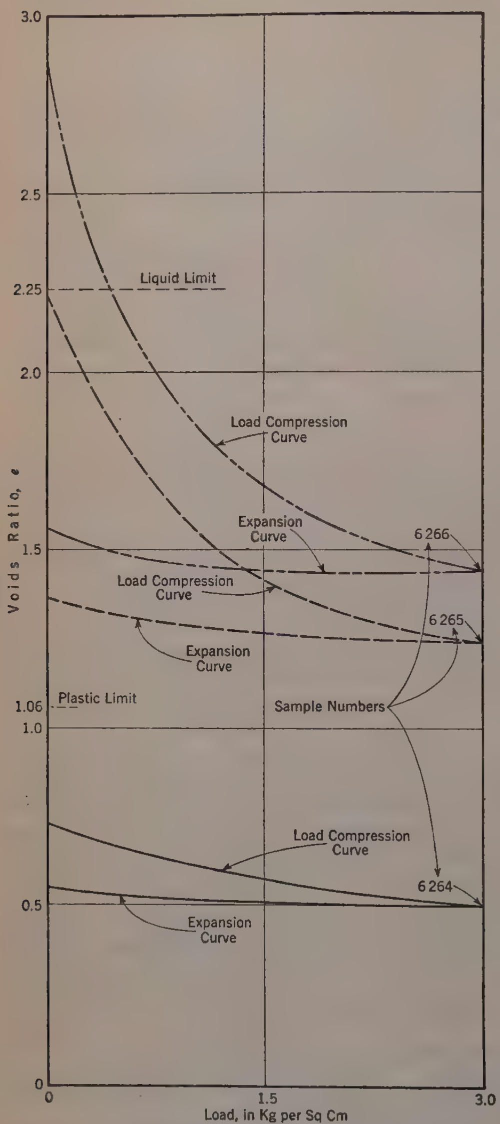


FIG. 4.—LOAD-COMPRESSION CURVES OF TWO MUD SAMPLES AND ONE SAND SAMPLE

(d) The writer was convinced that the underlying mud was still of nearly the same liquidity as was shown by the tests (this was later confirmed during excavation);

(e) It would be impracticable, for some years, to provide a concrete floor in the hangar;

(f) The weight of the seaplane, upon its beaching gear, would be about 50 tons, which must be supported safely; and

(g) The foundation design should be such as to allow the water contained in the mud, and in the sand fill, to work its way out slowly. Some of the borings at the site are shown in Fig. 7.

THE PRINCIPLES UNDERLYING THE FOUNDATION DESIGN

The following conditions were established by the writer as the basis for the foundation design:

(1) There must be a minimum number of main column foundations;

(2) The major footings must be stable against a lateral thrust of 2,100 lb per sq ft, at the center, and 700 lb per sq ft, average, on the projected diameter;

(3) The mud pressure in an excavation must be considered as nearly that of a liquid weighing 100 to 120 lb per cu ft;

(4) Every effort must be made to prevent inflow of mud into the bottom of an excavation from the surrounding area; otherwise, the greater part of the subjacent mud would be excavated and the surface overlying it would sink;

(5) No lateral support for piling must be considered, except in the cylinders which were to be backfilled with granular fill;

(6) Piling must extend to −60.0 ft before meeting firm bottom, and the old marsh material above this point would offer resistance to penetration, but

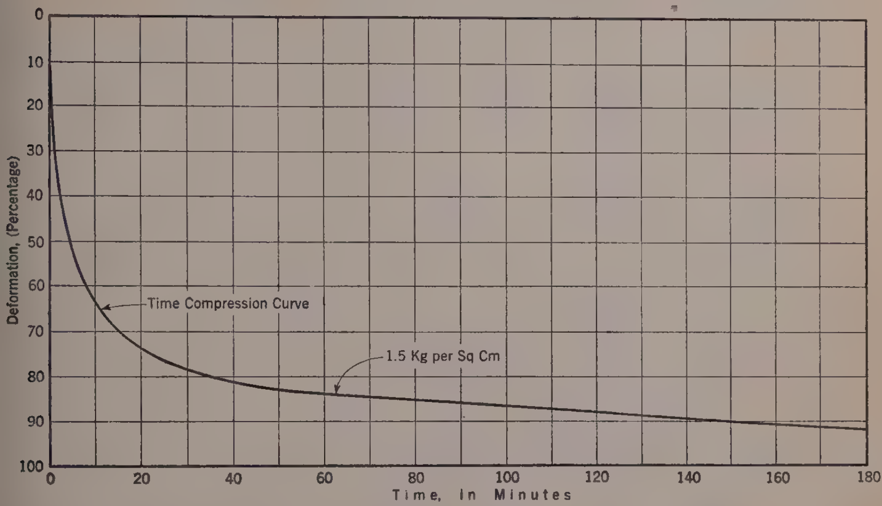


FIG. 5.—TIME-COMPRESSION CURVE OF ONE MUD SAMPLE (6265)

FIG. 6.—ERECTION OF HANGAR DOORS (THE DECAPITATED MUD WAVE IN THE FOREGROUND IS BEGINNING TO DEVELOP A NEW CRUST)

would provide a not-too-secure toe hold for the piling; hence, the piles must, if necessary, be jetted;

(7) The mud is a plastic—that is, it will stand shearing stress up to some (unknown) limit, but, beyond that stress, will flow like a viscous liquid, es-

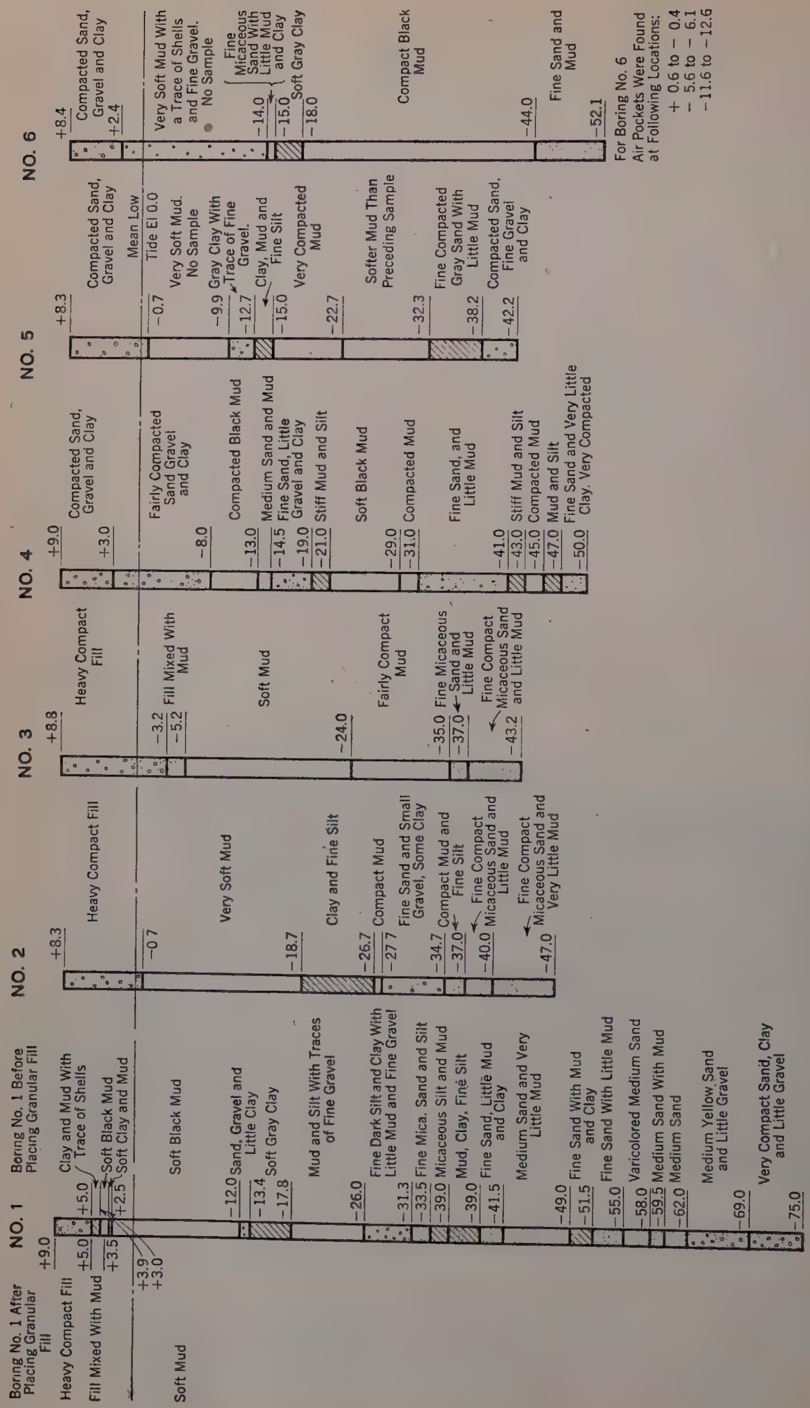


FIG. 7.—BORINGS AT THE TRANSATLANTIC SEAPLANE BASE

pecially when it is jarred as in the standard test for determining the liquid limit;

(8) The granular fill would provide ample lateral support at the surface, but the entire surface area in the vicinity might suffer translation;

(9) The weight of the seaplanes within the hangar, and of machinery loads, would cause a bursting pressure on the foundations; and

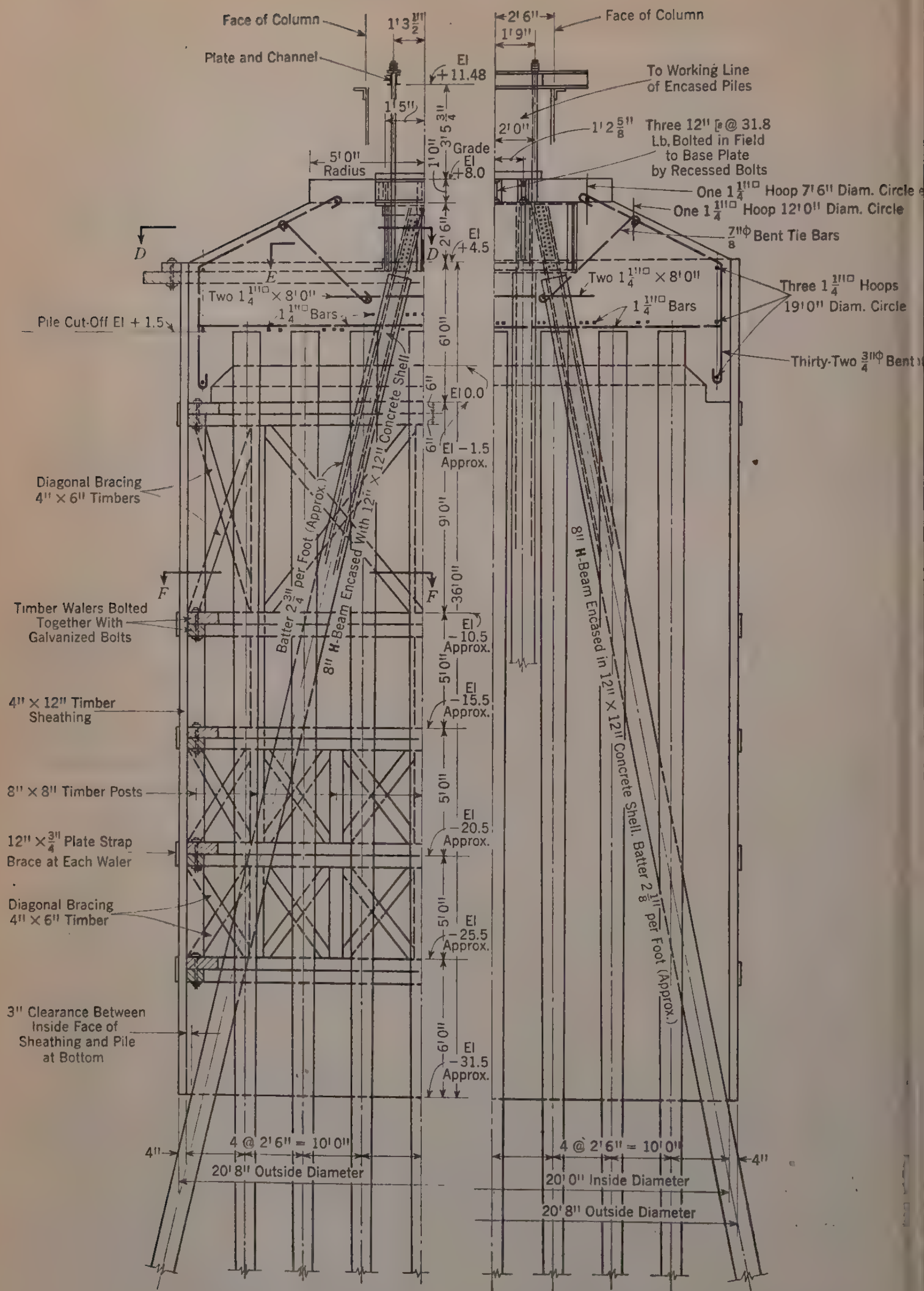
(10) As the mud slowly dries out below, and the surface settles, batter piles would be subjected to a lateral component stress; hence, only plumb piles should be used, except in the cylinders where there is granular fill placed down through water.

The average thrust of 700 lb per sq ft was determined from the equations of flow of a viscous liquid, at low velocity, around a circular cylinder, based upon an assumed shearing stress at the liquid limit; but there were no test data upon which to predicate this shearing value, other than the known angle of repose of the fill when it finally ceased to overflow the bulkheads. It was believed (and later confirmed) that at a short distance below the "crust" the fluidity was little less than when the tests were made in 1931.

Design of the Main Piers

Because of condition (1) it was advisable to use only four main columns for the hangar. In addition, however, layouts of the hangar and the planes to be stored in it determined that the maximum useful space, as well as the maximum of economy of construction cost, would result from the use of cantilever construction on all sides. The structure, therefore, is merely a four-legged table, with walls supported laterally against the roof. The rigidity against wind pressure, however, is diminished by such construction, and ample provision was made to permit the hangar to move (under wind and temperature) without binding upon the office building. The fixed walls of the hangar also were constructed with a slip joint at their upper support. To avoid the possibility that the hangar might sway in synchronism with wind gusts, its period of vibration was computed and found satisfactory.

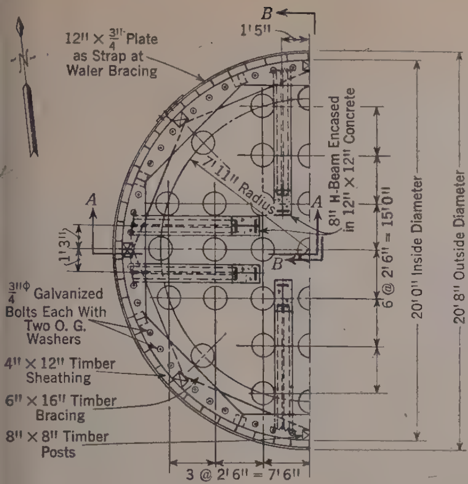
Because of conditions (3) to (9) the pier design shown in Fig. 8 was adopted. The cylinders were designed to be excavated, but not unwatered, thereby maintaining hydrostatic pressure inside, which would reduce the lateral inward pressure, and provide a maximum of downward pressure to minimize the inflow of subjacent mud. The wood piles were designed to withstand the column load as if they were braced laterally; hence, they were driven after the cylinder was excavated and were then supported by backfilling with the same granular fill that was being used for surfacing the airport. The cylinders were designed with ring and diagonal bracing to transmit the lateral thrust to their bottom and top. In order to provide for the bottom reaction the cylinder was sunk at least 6 ft into the old bottom of the river, and, for the top reaction, batter piles were provided. These were steel H-columns incased in concrete, battered as far as the size of the cylinder would permit, and stripped at the top and welded to the column grillage in the form of an A-frame. Eight such piles were driven, with steam hammers, forming four A-frames, two in each direction.



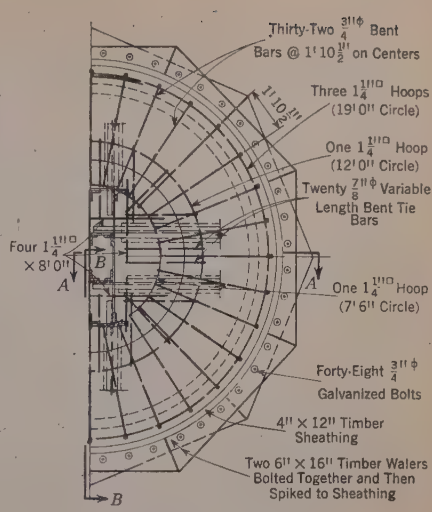
CENTER LINE SECTION *A-A* LOOKING NORTH

CENTER LINE SECTION *B-B* LOOKING WEST
Interior Timber Bracing Omitted. (See Section *A-A*)

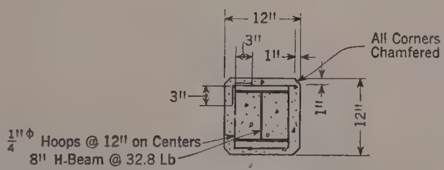
FIG. 8.—DESIGN OF FOUN-



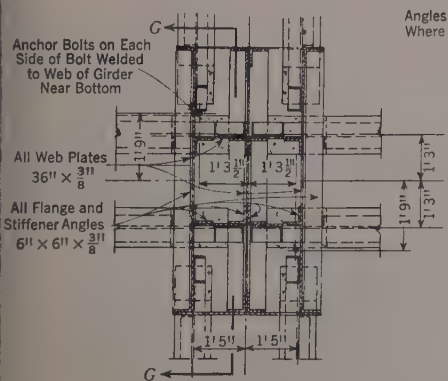
SECTION *F-F*



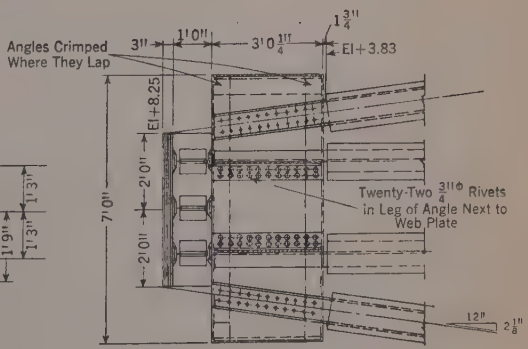
SECTION *D-D*



SECTION THROUGH CONCRETE PILES



SECTION *E-E*



SECTION *G-G*

DATIONS FOR MAIN COLUMNS

They were not counted upon for vertical load, although of course they carry their share. The wind thrust on the hangar is also resisted by them.

The sinking of these cylinders proved to be quite difficult. In addition to the buoyancy, considerable friction developed, first with the granular fill on the site, then with the drier mud crust below, and finally with the old river bottom. One cylinder required nearly 125 tons of surcharge, in addition to ring jets, to sink it and, in another case, greasing was necessary. In order to maintain the water level inside, it was necessary to supply water as fast as the bucket removed the mud, except of course when the jets were being used. This work was well handled by the subcontractors. Despite the precautions taken, a roughly circular area, extending as far as 15 ft out from the cylinder, sank 4 ft, or more, and was backfilled with granular fill.

Design of the Wall Foundations

The corners of the building were rounded to minimize eddies in the wind and, as a result, eddies are almost entirely absent even in strong winds. Because of the type of hangar construction, lifting doors were out of the question; and, because of the general design, round-the-corner doors were well suited (see Fig. 6). These doors are 10 ft 1 in. wide by 35 ft high, and weigh about 3 tons each, on two casters; yet they can be pushed by one man, although with difficulty. Another requirement of the foundation design, also, was that Pan American Airways required strength to support a 25,000-lb wheel load of the beaching gear, which might cross the wall foundation at any point within a considerable length. For lateral stability batter piles were considered, but were ruled out by condition (10); hence, plumb piles were used as shown in Fig. 3. Upon these piles is a hollow, rectangular beam, open at the top, across the top of which there are 6-in. I-beams to support the door rails. The hollow construction, aside from vertical, lateral, and some torsional rigidity, also permitted use of the foundation as part of the storm drainage system. Rain running down the doors, or falling on the nearby ground, passes into the channel within, and then, to prevent clogging with sand, the downspouts from the roof were led into the channel, thus providing ample water to keep the channel flushed out and free of obstruction. Between the rails, and supported on the I-beams is a 2-in. creosote-dipped plank flooring, neatly fitted to the rails to prevent air leakage. The rubber fabric weathering strips that are fastened to the bottom of the doors drag along these rails.

So long as the granular fill is undisturbed, these walls will remain in alinement. To provide for possible misalinement, the rails were merely clipped to the I-beams so that the rails may be shifted laterally on the foundation to a new line. To date (1941) there has been no motion. Vertical adjustment also was provided by not grouting the beams into the concrete until the roof structure had assumed its full load sag, under dead load only.

Along each side of the wall beam 1-in. round weep holes were provided to permit escape of water from the mud and from the fill, in order to simulate the experimental wicks so far as possible. As stated, the hangar floor and door rails are at El. +8.0, or just equal to the maximum storm tide ever recorded. To avoid possible water damage, the office building floor is at El. +9.0.

Design of the Office Building Foundation

The offices and public spaces are located in a two-story brick building having the form of a half octagon (see Fig. 9). To provide cylinders under this structure, for distributing the building loads, would involve great expense. It was

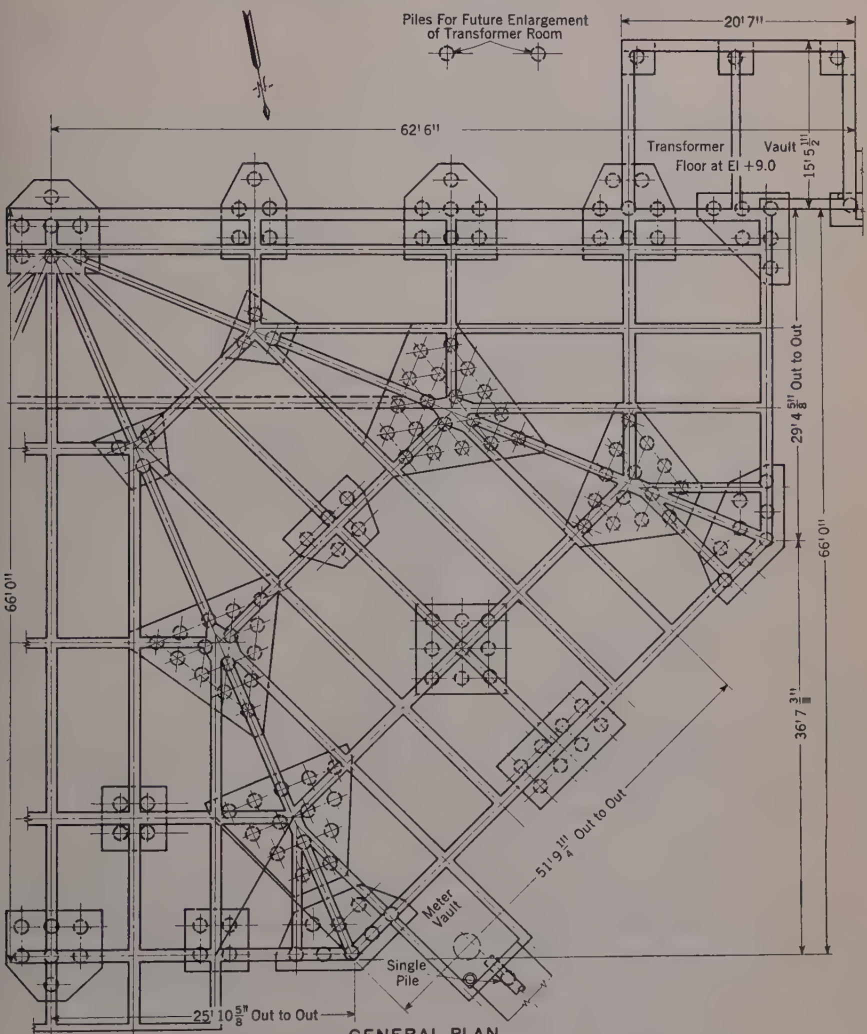


Fig. 9.—Design of Foundation, Office Building

decided that, with 10 tons per pile (because of the unsupported length in the mud), the number of piles required would be such as to give adequate lateral stability to the building. Later, during the driving of the piles, it was found that penetration could not reach −60, and some of the piles came to refusal at only 52 to 54 ft, and one or more at 48 ft. Therefore, the toe-hold capacity

of the old river bottom is at least as good as was assumed. It was impracticable to use the jet on these piles, because the area under the building was excavated to El. +5.0 (for an air space under the first floor), which space would have filled with water and drowned out all nearby work. At several footings under this building the underlying mud was exposed, and some of the excavations for them filled with mud which was exuded up through the bottom.

Design of the Hangar

There were no precedents for the design of the hangar. The Pan American Airways System stipulated the over-all dimensions and the clear height as 270 ft by 190 ft by 35 ft. They desired two rows of columns, giving two cantilever ends. The further restriction to four columns only permitted cantilever construction on all four sides, and also the saving that results in truss weight. Larger planes can be placed within, also, such that the trailing edge of the wing just touches the columns. The span of the wings thus can be greater than the column spacing; also, by entering askew, only one wing tip need clear a column (see Figs. 10, 11, and 12).

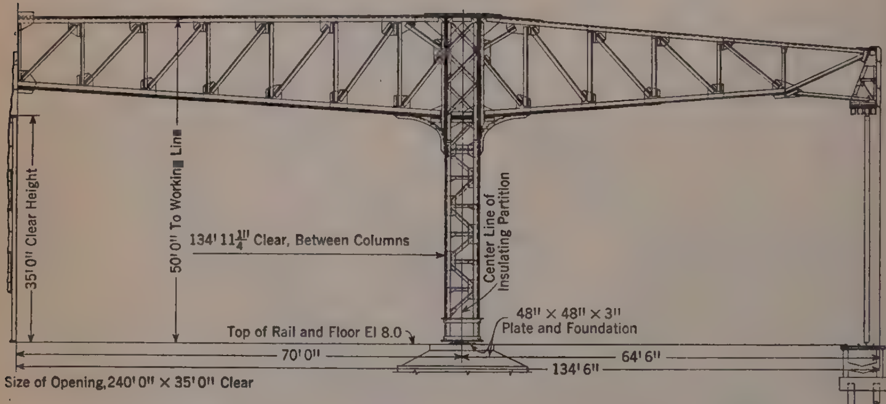


Fig. 10.—Design of Seaplane Hangar; Half-Section Showing Typical Truss

The unit load specifications,[3] in pounds per square foot, are as follows: Snow load, 30; roof uplift, 40; and wall loads, 40 on small areas, 30 on each door or wall panel, and 20 on the entire building. The 20-lb load acts at 135° to the larger wall, with wind also on the adjacent side, and the resultant wind acts at an eccentricity from the center of the building of 8% of the wall length, thus causing rotation.

The total wind was divided equally between the four columns, which were computed as though hinged at the base. (The torsion, also, was equally distributed.) They are 5 ft square, and bear on 24-in. steel sole slabs, which in turn bear on 48-in. slabs welded to the grillage. This 48-in. size was adopted to provide for tolerance in setting the grillage. However, the maximum

[3] "Using Aerodynamic Research Results in Civil Engineering Practice," by W. Watters Pagon, *Engineering News-Record*, October 31, 1935.

uncorrected error in the grillage positions was between 1 in. and 2 in. The trusses were computed for the column bending moment, and then curved knee-braces were added. There are two 3-in. anchor bolts, welded to the grillage,

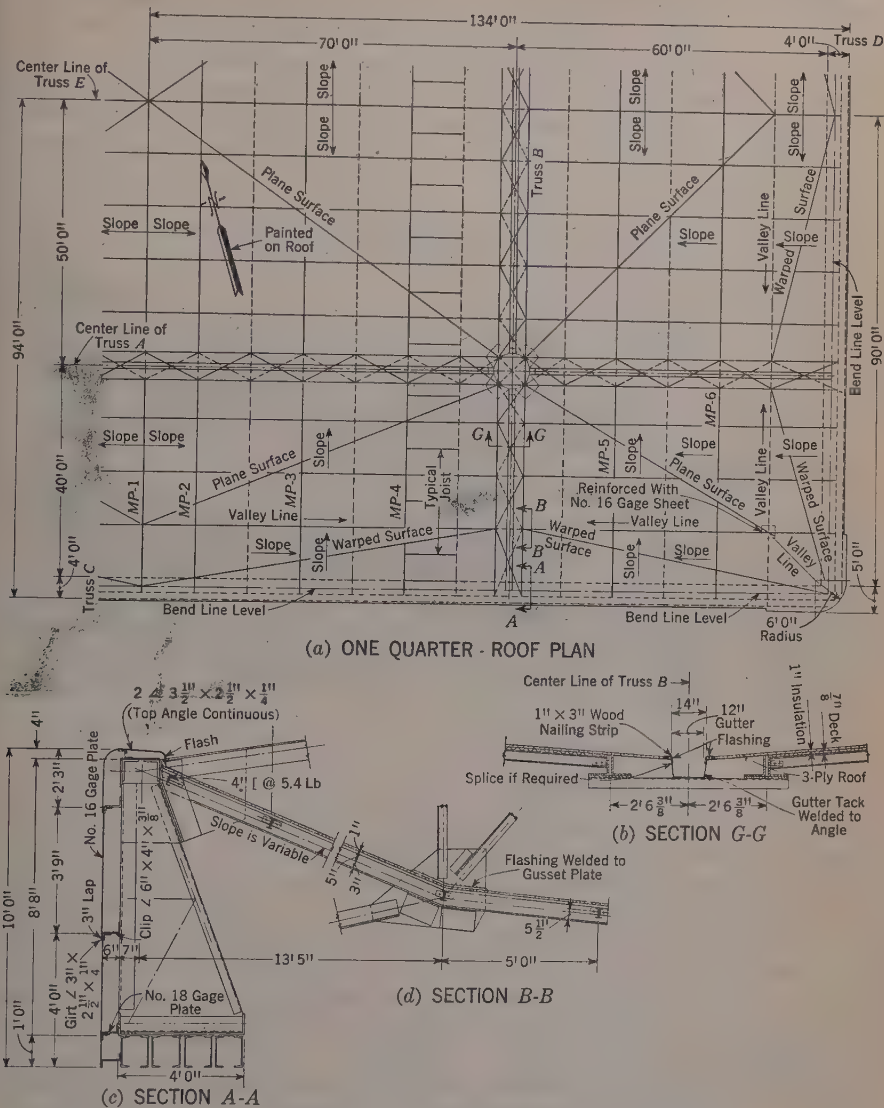


FIG. 11.—DESIGN OF SEAPLANE HANGAR; ROOF DETAILS

to resist uplift, because the building weighs less than the assumed uplift. During erection the columns were merely bolted down, but not restrained otherwise at the base; after all dead load strain was developed in the trusses, the bolts were slackened, but, no motion of the columns being found, the bolts were

again tightened and a 3-in. angle was welded to the bottom slab on all four sides to prevent sliding.

The upper tracks for the doors consist of 12-in. channels, facing each other, which are also computed as part of the chord section of the fascia trusses. These trusses are about 9 ft deep, of triangular section, and are braced by knees to the purlins to prevent torsion, although the vertical load is near the centroid. There is a 48-in. wide bottom plate, to which the four door tracks are riveted, and this plate and the channels form a horizontal girder to resist the wind thrust on the doors and on the fascia, in addition to acting as bottom chord (see Figs. 11 and 13).

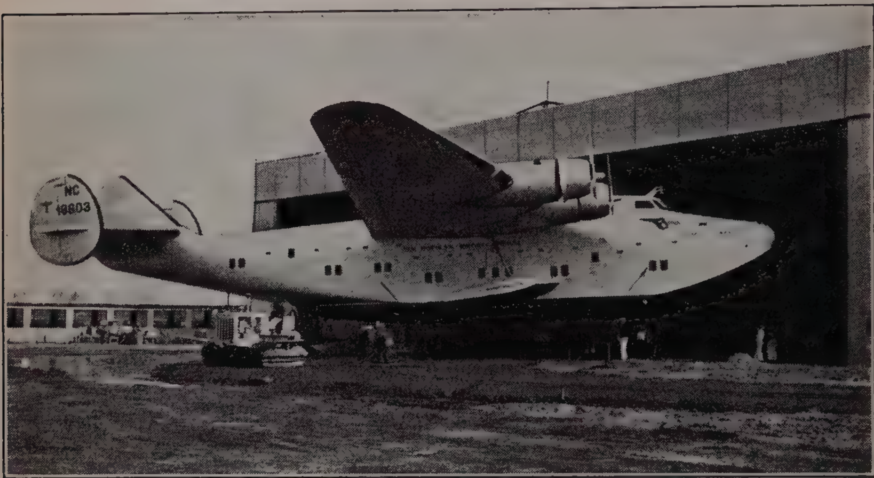


FIG. 12.—YANKEE CLIPPER ENTERING WEST DOOR OF HANGAR, FEBRUARY 26, 1939

The general structural design is shown in Figs. 10 and 11. When completed, the structure is indeterminate. It was computed as though determinate (with later corrections for continuity) and was so erected. The fascia trusses were carefully cambered so as to be truly straight and horizontal under full dead load, and the end connections between them, at the four corners of the hangar, were not installed until all dead load was in place. Continuity in the purlin knee-braces, however, prevented perfect vertical alinement of these fascia trusses, and a minor correction was necessary in the elevation of the bottom I-beams (which had not been grouted into the foundation) to make up for the discrepancy.

The end panel of the cantilevers was cambered upward for full dead load. Because of the cantilever construction there is much sag, or lift, in the fascia trusses. At the top of the doors provision was made for a sag, or a lift, of 3 in., due to snow or uplift (the computed distance was $2\frac{1}{4}$ in.). For plumbing the doors an adjustment of $\frac{3}{4}$ in. was made in each caster. The doors and hardware are larger than any standard design; hence, they both were computed and designed especially for this structure (see Fig. 13).

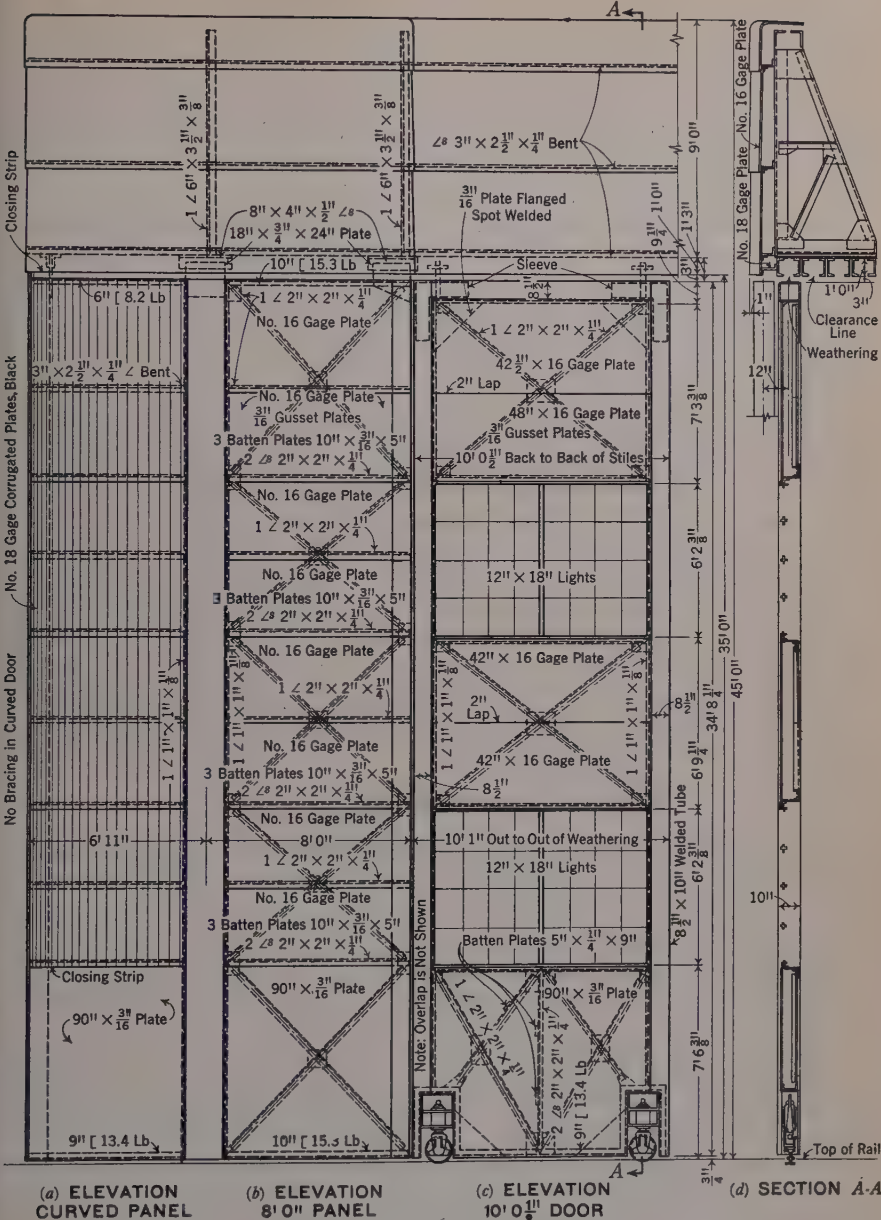


Fig. 13.—Design of Doors

The roof structure is unusual. For three reasons it is inverted, with the roof supported upon the bottom chord of the main trusses and purlins. The first reason (historically) was roof drainage, there being no place for downspouts except at the four columns. (The building corners are so designed that they may be put on casters later, if future wing spans require greater useful opening.) The other, and more determining reasons, are the protection of the structure from inside fires, and the reduction in the height of the space to be heated. A ceiling would have added a considerable percentage to the total load. The roof-deck design offered many problems, but they were successfully met as shown in Fig. 11. At first there were a few leaks in the flashing collars around the gusset plates of trusses and purlins, but when these had been corrected there was no further trouble. The deck is of deep-corrugated steel, covered with 1 in. of insulation and a 15-yr ply roof covering.

The night lighting of the hangar was solved by placing nine 1,500-watt lamps in open, porcelain, enameled reflectors at each column head, and six at each of the 190-ft ends. Since the walls are insulated and painted white, and the roof is galvanized, the illumination is very nearly uniform. The lighting load is 72 kw, or about $1\frac{1}{2}$ watts per sq ft. The electrical system centers in a transformer vault in a brick room within the hangar. Even then, oversize copper was required for some circuits in order to hold the maximum voltage drop to 5 volts, under full load conditions.

In order to eliminate piping runs under the floor, where settlement might cause breaks, all steam and return piping is at the ceiling, and the system operates at sufficient pressure to lift the condensate from the floor type of unit heaters and return it to the boilers. The floor is a bituminous concrete, placed on a sand fill which was thoroughly wetted and rolled with a steam roller, and has given perfect satisfaction.

Design of the Office Building

The office building houses all the operating and traffic personnel, and also a portion of the division engineering personnel (the remainder are in small shop offices in the hangar). There is space for tickets, facing the Concourse, and on the other side are rooms, in series, for Public Health and Immigration inspection. In the rear of the Concourse is the customs inspection counter (see Fig. 9(*b*)).

Landing, Beaching, and Fueling Facilities

Passengers are landed at a float and pass over a floating walkway and then a pile trestle to a covered walkway. This walkway leads directly into a waiting room at the entrance to the federal quarters. The pump house at the end of the walkway provides gasoline either on the float or at the head of the beaching ramp. There is underground storage for 16,000 gal of gasoline.

Shortly before completion of the hangar it was decided, as an additional precaution, to install a standard-gage railroad track leading into the south and the west doors. No trouble was experienced, however, except at the mud wave south of the hangar, where the track deflected under the weight of a 10-ton

steam roller by about 6 in., over a length of about 20 ft. This was finally corrected.

Operations from the Seaplane Base

The first seaplane arrived at the airport on November 17, 1937. Because the facilities were incomplete, space was leased in the Curtiss-Caproni building as a hangar for ships and for division and traffic personnel. In December, 1938, the city made delivery of the entire project to the Pan American Airways System. During the temporary operation period the planes were beached on the small, public ramp No. 1 (see Fig. 2).

On March 16, 1938, air mail and passenger service to Bermuda was commenced by Pan American and Imperial Airways. During the winter this was direct; in other months there was a stopover at Port Washington, Long Island, but now (1940) the service is direct from La Guardia Airport or from Baltimore. Service to Europe commenced on May 13, 1939, on the route, Baltimore—Port Washington—Bermuda—Horta (Azores)—Continental airports, or via Botwood Harbour, Newfoundland, and in summer now operates from La Guardia Airport. All maintenance and repair work is performed at the Baltimore base in winter.

Landplane Base

The airport, with the buildings for landplane use, is expected to be in operation in 1940. Adjacent to the air station will be three hangars. A Maryland National Guard Unit will occupy the east corner of the field (see Fig. 2).

Cost

It is contemplated that all of the work on the airport that is now planned will be completed for a total of about $6,500,000, including stabilized runways and one of the hangars.

Personnel

Work on Stage Three of the airport was started under Mayor Howard W. Jackson and Chief Engineer Bernard L. Crozier, who was succeeded at his death by Frank K. Duncan, M. Am. Soc. C. E. Work on a portion of the fill, the beaching, and landing and fueling facilities was under Harbor Engineer Frederick M. Kipp, Jr., Assoc. M. Am. Soc. C. E.; work on the buildings was under Buildings Engineer William A. Parr, and immediately under Assistant Buildings Engineer Martin Koenig, Jr. Delano and Aldrich were consulting architects on the office building; Henry Adams, Inc., were the consulting engineers on the heating system. The writer, as airport consulting engineer, designed the airport facilities and layout and the buildings, etc. In the second stage Bancroft Hill served with the writer as consulting engineer. Capt. L. L. Odell, chief airport engineer for the Pan American Airways System, approved the plans for the company. Henry L. Shryock, Jr., was project engineer for the PWA. The contractors were as follows: For the fill, The Arundel Corporation; for foundations, Consolidated Engineering Company, under whom was the Raymond Concrete Pile Company on the main piers and the piling; for the hangar, Kaufman Construction Company; and for the office building, W. E. Bickerton Construction Company.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
REPORTS

CONSOLIDATION OF EMBANKMENT AND FOUNDATION MATERIALS

PROGRESS REPORT OF SUB-COMMITTEE NO. 2 OF THE COMMITTEE OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION ON EARTH DAMS AND EMBANKMENTS

1. General

The desired degree of compaction required for the soil being used in a dam having been determined, attention must be directed to those factors which affect the means by which such degree of compaction may be attained. These are: Nature of the soil, moisture content, thickness of layers, pressure of roller feet, arrangement and spacing of roller feet, weight of tractor, and number of passes of compacting unit.

It is evident that a large number of combinations of these factors are possible. Sufficient information is not available to coordinate the effects of each completely. However, the purpose of this Report is to present an account of present theory and practice and a description of the relative effects of the various factors which have already been established.

Succeeding reports will endeavor to keep abreast with changes in theory and practice. For this purpose, the Sub-Committee requests that it be supplied with all new information which readers may obtain by experimentation or during construction. The Sub-Committee will then act as a clearing house for such information with, it is believed, considerable benefit to the profession.

2. Factors Affecting Control Density of Soils

(*a*) *General.*—In many cases the materials available for the construction of an earth dam are limited in type. In other cases some choice is available. In all cases it is necessary to utilize those materials which will result in the least cost of a satisfactory structure. The problem frequently revolves about the question of economy, involving the use of the cheapest materials with a fat cross section or more expensive materials with a lean cross section, unless a minimum section is fixed by the strength of the foundation.

Tests of available materials, analyses of alternative designs, and cost estimates will finally result in a definite choice of type of dam, of the several types

Note.—Written discussion of this Report will be transmitted directly to the chairman for the information of the Sub-Committee.

of materials which will be used, and definite densities which are necessary in various parts of the dam to conform with the assumptions used in the design. The embankment materials should then be compacted to such densities. These are the control densities which govern the compaction during the progress of the work.

The density of soils which, according to laboratory and field tests, will insure the characteristics desired, should be determined after consideration of the following critical criteria: Shearing strength, settlement characteristics, permeability, plasticity, shrinkage characteristics, expansion characteristics, and critical density.

(*b*) *Shearing Strength.*—A reasonable amount of compaction is required to insure the necessary shearing strength for an economical dam. A typical relation between the effort expended in original compaction and shearing strength is indicated in Fig. 1 for a constant normal loading. It is obvious that there is some degree of energy expended in original compaction, such as point A, beyond which the cost of a greater amount of compaction would not be commensurate with the saving resulting from the possible use of steeper slopes.

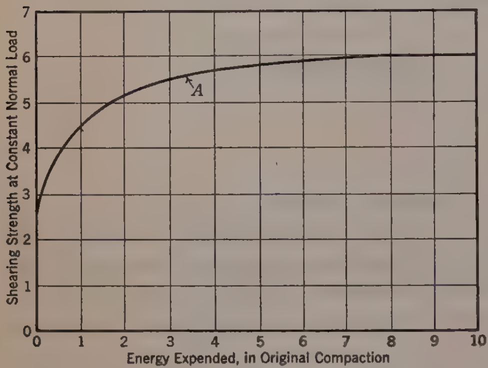


FIG. 1

In some cases, the slopes of the dam are governed by the necessity of a wide base to insure stability of a weak foundation. In this event, a very high degree of shearing strength is not required and point A could be chosen lower on the curve. Thus, the control density required to meet the criterion of shearing strength is established. It is nearly always the case that the required control density to obtain the necessary shearing strength and better than critical density, as explained later, will satisfy the minimum limit of density required for other criteria. However, there is a desirable maximum limit to compaction of cohesive materials as will be shown subsequently.

(*c*) *Settlement.*—As mentioned heretofore, the necessary compaction to obtain the density required for the shearing strength used in the design seldom, if ever, must be increased to insure against excessive settlement after construction. However, the necessary overbuilding of some dams, to compensate for anticipated settlement, is expensive. To estimate such settlement requires laboratory consolidation tests and settlement analysis. At present only approximate estimates can be made to determine expected settlement, since the problem is frequently very complex. It is possible that more exact analyses can be made after sufficient correlation is obtained of actual and theoretical settlements.

Therefore, this Sub-Committee strongly recommends that complete settlement analysis be made for every embankment to be constructed and that a

comparison of estimated settlement with subsequent actual settlement obtained from gages be forwarded to this Sub-Committee for use in later reports.

(d) *Permeability.*—The coefficient of permeability of an impervious embankment material seldom varies significantly with density, in the range generally considered in embankment construction, and therefore the compaction required for water tightness is never critical. It will be shown later that very pervious cohesionless soils drain so rapidly that difficulty is experienced in keeping them wet enough for best compaction.

(e) *Plasticity.*—A certain amount of plasticity in cohesive soils is desirable in a dam so that it can conform to unequal settlement in the foundations and also to the differential settlement of the dam itself between the center and the sides, particularly if there is a sharp break in the longitudinal profile. Plasticity, of course, decreases with an increase in compaction. However, it is believed that there is sufficient inherent plasticity in ordinary cohesive soils for ordinary conditions. At the present time, the handling of extraordinary conditions is subject only to judgment.

(f) *Shrinkage.*—Certain clayey materials have a habit of cracking badly, at the surfaces of dams, when they dry out. These cracks are sometimes several inches wide and several feet deep. The remedy is to cover such materials with a coat of coarse material, to decrease the height of capillarity, and to reduce evaporation. Otherwise, heavy compacting with relatively little water is the alternative. In some dams this condition has required a supply of water to a ditch on the top of the dam to prevent the material from drying out during dry seasons.

(g) *Expansion.*—When a cohesive soil is heavily compacted under an initial water content, which leaves it after compaction in a partly saturated state, some expansion tendencies are locked up by the forces of capillarity. When such capillarity is destroyed by saturation due to rain or reservoir seepage, the material expands somewhat unless the expansion pressures are resisted vertically by the weight above it and horizontally by shearing resistance of the foundation.

Fig. 2 shows the result of experiments made at the U. S. Engineer Office, Denison, Tex., to determine the expansion pressures of material having the following characteristics:

Size in millimeters	Percentage of material finer than
0.002	22
0.005	31
0.020	46
0.050	68
0.100	86
0.300	100

The compaction characteristics of the soil used in the test are shown in Fig. 3.

The expansion tests were made in ordinary consolidation apparatus, maintaining a constant original volume of soil as distilled water was permitted to enter and be absorbed by the soil sample. Small volume changes, measured

by a dial indicator, indicated when load was required to stop expansion. Tests were considered complete when no further expansion tendency was indicated, and this was determined from a load-versus-elapsed-time graph plotted on semilogarithmic paper as the tests progressed.

It is extremely difficult to get accurate results from tests of this kind. However, sufficient information can be obtained to indicate conclusively that the greatest expansion obtains with (1) the greatest compaction, and (2) the least original water content. Of the most importance is the necessity for the greatest possible water content consistent with desired compaction, and Fig. 3 shows that, for the material under test, 90% of complete saturation can be obtained by keeping always on the wet side of optimum moisture.

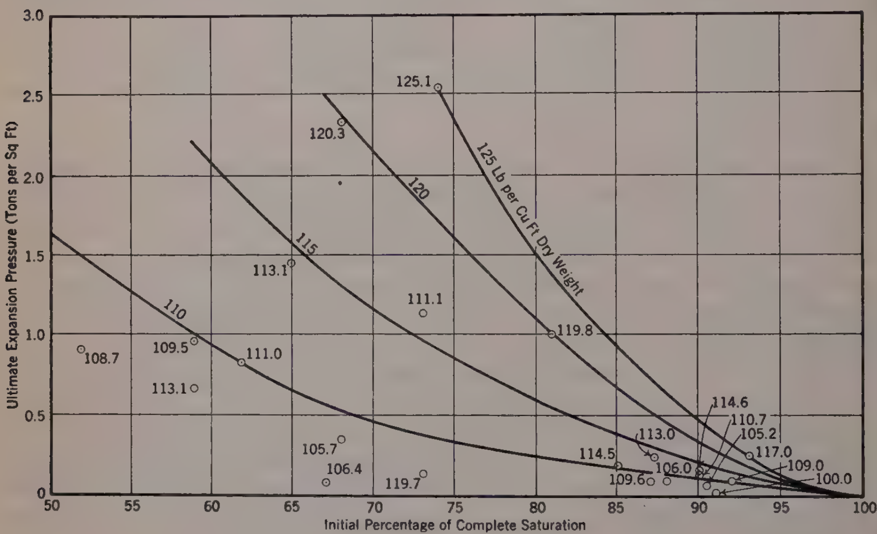


Fig. 2

However, despite all precautions, some expansive forces will exist in cohesive soils although in most cases they may be negligible. Although the necessity for a limit to the amount of compaction may be the exception rather than the rule in dam construction, in some cases this feature embodies a positive danger and is a governing factor in dam design. The partial failure of at least one dam has been attributed to expansion of heavily rolled soils. The matter deserves investigation for every dam.

The permissible force of expansion is difficult to determine. The vertical compressive stresses available to resist vertical expansion are not equal to the weight directly above. Moreover, the great danger is the tendency for horizontal expansion, the building up of expansive forces, and the sudden release of them due to failure along some nearly horizontal plane in the dam or at the surface of a weak foundation.

Research is greatly needed on this phase of design. At present engineers have for a guide only the rule-of-thumb which specifies that the material at any

point in the dam shall not be compacted to a greater density than that which, as shown by laboratory tests, will correspond to an expansive force greater than the weight of material directly above it.

Obviously, this would theoretically require zero compaction at the surface of the embankment which actually must, of course, be compacted to a reasonable strength. If expansion occurs near the surface, the damage would be local and must be taken care of by maintenance. However, when available, a sufficient layer of non-expansive coarse material should be placed on the outside of the fill to hold it in place. Where the material for backfilling retaining walls is of an expansive nature, it should be compacted as lightly as feasible and with maximum initial water content. Even for dams composed

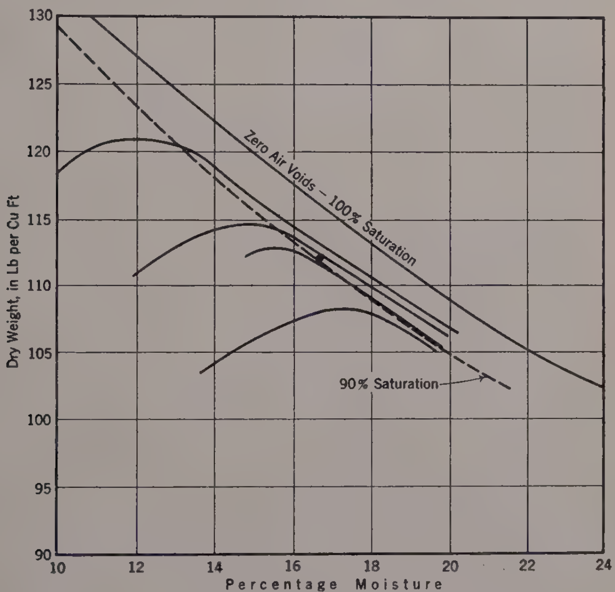


FIG. 3

entirely of cohesive materials it is sometimes possible to route the more expansive material to the central part of the dam and the less cohesive to the outside and top.

(*h*) *Critical Density.*—For years engineers have observed flow slides and quicksand conditions in saturated cohesionless soils. Although they have generally associated such slides with uniform, fine-grained materials, coarse-grained materials of this type have also been known to liquefy and flow when disturbed. They have also known for a long time that cohesionless materials in a loose state are potentially dangerous, whereas compact materials are generally stable. However, it has been only during recent years that an attempt has been made to tie the thing down to a quantitative basis and to devise a means of testing which will enable the designer to get at least an idea as to whether the particular material with which he is dealing is likely to give trouble.

It has been determined that, for a loosely compacted cohesionless material, the volume at failure is less than the initial volume, whereas, for a densely compacted sample, the volume at failure is greater than the initial volume. When such material is saturated, any decrease in volume of a loose soil must be accompanied by an expulsion of some of the water out of the voids. Conversely, any increase in volume of a dense soil must be accompanied by a drawing of water into the voids. When the soil is not saturated, or when the application of the shearing stress is so slow that the pore water will have ample time to exit or enter, corresponding to changes in voids, the strength of the material will be unchanged. However, a dam or a foundation may be subjected to a shearing force so suddenly, as in the case of an earthquake or other disturbance, that the pore water will not have time to adjust itself to changes in volume of the soil. In such cases, the tendency of a loose saturated soil to decrease in volume during sudden shear will allow insufficient time to expel the pore water, and some of the internal load is temporarily carried by the water, thereby reducing the shearing strength of the material. If the remaining shearing strength is insufficient to maintain equilibrium, a flow slide results. On the other hand, the tendency of a dense saturated soil to expand during sudden shear will set up negative stresses (that is, a partial vacuum) required to draw the water into the voids, and will increase the internal load on the soil, thereby increasing its shearing strength.

"Critical density" may be defined as that density of a cohesionless soil which, when compacted into a dam, will tend to have the same volume at ultimate shearing stress as it had originally. Thus, when saturated, a cohesionless soil which is looser than critical would tend to decrease in volume when sheared and would lose strength, when sheared suddenly, as previously explained; whereas such a soil which is denser than critical would tend to increase in volume and attain greater strength.

Certain tests have been devised to determine the critical density of cohesionless soils. The most recent tests make use of what is known as the triaxial machine.[1] However, although such tests are useful in forming an idea as to probable trouble, the best of them give no assurance that they truly represent the conditions in nature, and their use should be tempered with judgment and a full knowledge of their limitations.

A critical-density test program has been inaugurated by the U. S. Army Engineers and the testing work is now in progress at Harvard University and Massachusetts Institute of Technology, at Cambridge, Mass., and the U. S. Waterways Experiment Station, at Vicksburg, Miss. It is realized, however, that the answer cannot be obtained entirely from laboratory tests, but that large-scale field tests must be used to check results. In this connection a large-scale field test was conducted recently for the Mill Creek Dam (Washington) by the U. S. Engineer Office, at Bonneville, Ore., the results of which have not yet been reported.

It is the general opinion that earthquakes are not the only phenomena that can cause the sudden application of the shearing stresses which require satu-

[1] "Symposium on Shear Testing of Soils," A. S. T. M., Vol. 39, 1939, with particular reference to the papers by J. D. Watson and D. W. Taylor, Assoc. Members, Am. Soc. C. E.

rated cohesionless material to be more dense than critical. Rapid drawdown of the reservoir, rapid construction, local slips in the embankment or foundation, blasting in the vicinity, the passage of heavy trains or vehicles, and sudden loading of the top of the embankment all constitute hazards. For these reasons, the probability of earthquakes should affect the choice of factor of safety rather than the choice of general criteria of compaction.

All evidence indicates that saturated, cohesionless fine-grained soils with great uniformity of gradation are to be viewed with suspicion since this is a characteristic which is present in most actual flow slides. Coarse, well-graded soils present the greatest ease of attaining required density. As far as the Sub-Committee knows now, no trouble with flow slides has ever been experienced with cohesive materials, or with well-graded materials or, in dams, with coarse, free-draining materials, although rock slides of the flow variety are not uncommon in glacial areas, such as the high valleys of the Alps.

Tests show that the critical density becomes greater as the vertical pressure increases. Therefore, greater density is required for high dams than for low ones. In this connection, it is quite possible that, for a given cohesionless soil, there is a limit to the height to which compacted cohesionless soil can be placed if critical density is desired.

It is the consensus of opinion that all cohesionless soils in the saturated portions of dams above the foundation should be compacted to a greater density than critical, not only to provide a factor of safety, but also to compensate for additional hazards sometimes present during construction.

If a dam is constructed so rapidly that it or its foundation does not have sufficient time to drain—that is, sufficient time to expel the pore water corresponding to consolidation due to loading—the trapped water constitutes a source of weakness. The result is that, during construction, the dam (or foundation) has only a part of the shearing strength which it will have eventually, when drainage is complete. Hence it is necessary to carry the compaction to a density sufficiently below critical so that the tendency to contract due to residual compression will be counterbalanced by an increased tendency to expand due to shear.

In some cases this additional compaction will be found to be impracticable or uneconomical. In that event, a more conservative design, reducing the intensity of shearing stress in the dam and foundation, is indicated.

The compaction of materials to a greater density than critical is easy to accomplish in most rolled-filled dams. At Franklin Falls, N. H., it was found that the compaction of the cohesionless embankment to a density equal to 80% of the maximum density obtainable with dry material in the laboratory would result in a state sufficiently denser than critical. This degree of compaction appears rather high and may not be necessary in other types of cohesionless soils.

The shells of hydraulic-fill dams, composed of fine-grained, uniformly graded sand, may require compaction to attain better than critical density, particularly if they rest on foundations which have a low factor of safety. Such compaction would have to be done by rollers, tractors, or other mechanical devices and would constitute considerable expense and interference. The result may mean

the abandonment of that type of construction with such materials, on the ground of expense.

Cohesionless foundation soils have been compacted successfully by explosives at the Franklin Falls Dam. However, it has not been demonstrated that this method of compaction will prove successful at other places.

The necessity of removal of cohesionless foundation soils which are less dense than critical depends on their extent and location. Large deposits of such materials must be consolidated or removed. However, a well-compacted embankment can bridge over local pockets of objectionable foundation materials which are temporarily deficient in strength. This applies particularly to places where such pockets are not in the zone of critical loading. Only in localities where violent earthquakes may reasonably be expected must such pockets be removed.

It has been previously mentioned that a saturated, cohesionless soil, having a density which is less than critical, loses strength when subject to sudden shearing forces but it does not lose necessarily all of its strength.

Designing on the basis of residual strength is feasible provided engineers are confident of their ability to determine it. Unfortunately, methods of determining residual strength, at the present time, are very questionable.

However, in borderline cases, the slopes of the dam have been flattened to reduce foundation stresses.

It is frequently possible, by careful selection of borrow-pit materials, to route the worst of the cohesionless soils to those parts of the dam which will not be saturated.

3. Methods of Compacting Soils

(*a*) *General.*—In the compaction of either cohesionless or cohesive materials, the exact type of equipment, number of passes, and thickness of layers cannot be predetermined accurately without experimentation with test fills. For work of magnitude, using many units of compacting equipment, pre-experimentation with test fills usually will bring ample returns since, without them, the adopted equipment might have to be remodeled or abandoned later.

(*b*) *Equipment, General.*—For cohesionless materials, it has been demonstrated conclusively that the character of the material and the required consolidation density affect enormously the necessary type of equipment.

The normal type of equipment, for compacting cohesive material, is a crawler type of tractor drawing two or more sheepsfoot rollers. No superior type of equipment has been developed. Smooth rollers confine their greatest compaction at the surface, requiring thinner layers and providing a smooth instead of a rough bond between layers. Grooved rollers for cohesive materials have not been used to a sufficient extent to demonstrate their value.

As the materials being compacted become coarser and lose their cohesive properties, the effect of the roller decreases and the effect of the vibrations of the tractor increases. For purely cohesionless materials, the tractor has much the preponderant effect. When the grading of the material approaches that of fine concrete aggregate, the rollers are not effective at all. For this and coarser material, even the tractor loses its effectiveness, so that considerable compaction is impossible with present methods.

(c) *Compaction of Cohesionless Soils.*—Of the compaction tests made in the United States, the most extensive series are those made by the U. S. Engineer Office, Boston District, on the cohesionless soils for the Franklin Falls Dam.[2] The results of these tests and other less extensive investigations by the Army Engineers, in connection with that dam, are summarized in Tables 1 and 2.

TABLE 1.—TYPE OF MATERIALS TESTED

Description	Materials[a]			
	A	*B*	*C*	*D*
Type of material	Fine Silty Sand	Medium Fine Sand	Well graded Silty Fine to coarse sand	Coarse Well graded Sharp sand
Percentage passing 200 sieve	55 to 75	11 to 14	15	5
Percentage retained on ½-in. sieve	0	2	15	10
Effective size, in millimeters	0.02	0.07	0.06 ±	0.09 ±

[a] *A* and *B* as in the Franklin Falls report; *A*, *B*, and *C* were compacted during the same summer and with the same equipment.

TABLE 2.—TYPE OF EQUIPMENT USED

Item No.	Type	Total weight (lb)	Unit weight (lb per sq in.)	Pressure contact assumption
	(*a*) SINGLE ROLLERS USED FOR MATERIALS *A*, *B*, AND *C*			
1	Tractor	29,450	8.7	....
2	Smooth roller	7,836	25.6	6 in.
3	Disk roller	19,900	98.4	9 in.
4	"Large-foot" sheepsfoot roller	4,185	84.5	1 row of 4 ft
5	"Small-foot" sheepsfoot roller	4,858	115.0	2 rows of 4 ft
6	Altered sheepsfoot roller[a]	....	230.0	1 row of 4 ft
	(*b*) DOUBLE ROLLER USED FOR MATERIAL *D*			
7	Tractor	41,500	7.2	....
8	Double roller	9,700 (total)	220.0	1 row of 8 ft

[a] Same as No. 5 except every other horizontal row of feet removed; feet were not staggered.

The conclusions drawn from the Franklin Falls original and supplemental tests were as stated in the following paragraphs, (1) to (9).

(1) Tractor Alone.—It was found that the use of a tractor alone gave results which were inferior to those obtained from the combination of the tractor with the disk or sheepsfoot rollers when used for the fine and medium materials, *A*, *B*, and *C*; but with the very coarse material, *D*, it was demonstrated that the tractor alone gave the same compaction as with a tractor-drawn sheepsfoot. However, with the coarse material, such compaction was not great and, in order to obtain a high degree of compaction, either a different rolling procedure, heavier rolling equipment, or basically different methods must be used than have been used in previous tests.

[2] "Compaction Tests and Critical Density Investigations of Cohesionless Materials for Franklin Falls Dam," U. S. Engineer Office, Boston, Mass., April, 1938.

It is believed that, even in fine cohesionless materials, a very heavy crawler tractor has the preponderant direct effect on compaction, which is accounted for by the vibration transmitted from the motor through the treads. Although rollers are of limited assistance for direct compaction on cohesionless materials, the pockets made by them assist indirectly by providing a means for the necessarily large quantity of water to get into the finer materials. Material *D* in Tables 1 and 2 was so coarse that the rollers were not necessary to get the water into it.

(2) Rollers (General).—Although rollers give relatively less direct compaction than the tractors, the added assistance to direct compaction which they do give, in well-graded, silty, cohesionless materials, as well as in fine and medium cohesionless material, warrants studies to determine the most suitable type.

(3) Smooth Roller.—The tractor-drawn smooth roller produced relatively high compaction at the surface but less near the bottom than with other equipment, or even with the tractor alone.

(4) Disk Roller.—For materials *A* and *C*, Table 1, the tractor-drawn disk roller gave results that were slightly better than those obtained by the tractor-drawn sheepsfoot rollers for the 12-in. and 18-in. layers. With the sheepsfoot roller on layered embankments, the top of the layer surface was rather broken by kneading action and remained comparatively uncompacted under overlying layers, giving a jagged appearance to the depth-compaction curves, whereas the disk roller apparently did not produce such irregularities in these curves.

However, it will be noted that the disk roller weighed about four times as much as the sheepsfoot rollers. This has two effects. It not only tends to add compacting weight to the lower part of each layer, but principally, because of heavier hauling duty, results in greater laboring and vibration of the tractor —that is, the tractor accomplishes more by the additional vibration and shear effect produced by having to draw the heavier disk roller. Thus it was not demonstrated that a disk roller will give the same compacting effect as a sheepsfoot roller of the same weight. However, the disk roller was designated as one of two types of acceptable rolling equipment at the Franklin Falls Dam.

(5) "Large-Foot" Sheepsfoot Roller.—The "large-foot" sheepsfoot roller was abandoned as its results were inferior to the small-foot sheepsfoot roller.

(6) "Small-Foot" Sheepsfoot Roller.—The "small-foot" sheepsfoot roller was found more satisfactory than the two other types investigated and was adopted for one of the two permissible types for the Franklin Falls Dam.

(7) Altered Sheepsfoot Roller.—This type of roller was obtained by removing every other horizontal row of feet from the "small-foot" sheepsfoot roller. This resulted in double the unit feet pressure since only half the number of rows of feet were in contact.

The results obtained by this roller on material *A* were not materially different from those obtained from the "small-foot" roller. Thus, although the unit pressures were doubled, the coverage was only one half for the same number of passes. Also the greater spacing of the feet resulted in less kneading action.

Moreover, since alternate rows of feet of the "small-foot" roller were staggered, the removal of every other row left the altered roller without staggered feet. This also reduced the kneading action. This type of roller was abandoned for use at the Franklin Falls Dam.

(8) Tampers.—Tamping equipment has not been used as the principal means of compaction of earth fills on any large projects in the United States, but several have been designed and used in Germany. Their principal drawback so far is lack of ruggedness. Reports indicate that the machines shake themselves to pieces while shaking down the fill.

Two types of tamping equipment were tested at Franklin Falls, one consisting of a drop-weight tamper weighing 2,164 lb and falling 18 in., and the other a pile-driving air hammer, specially mounted. Although these units were believed to be indicative of those attainable with practical equipment designed on the same principle, the results of tests on material *A* were inferior to those obtained by either rolling equipment or vibration. The ordinary penumatic hand tamper has been used successfully, of course, for all types of material in confined spaces not capable of being reached by rollers.

Also, for places inaccessible with rollers, a type of domestic hand tamper, consisting of a simple gas engine with tamper feet, has been used successfully in the United States for similar conditions. This equipment is self-contained and does not require leads to the original source of power. It can be operated by one operator but the use of two or three men was found most economical on San Gabriel Dam No. 1. Foreign equipment of this type has not proved successful.

(9) Vibrators.—Although experience with vibrating machines in Germany has indicated successful possibilities for the compaction of cohesionless soils, similar equipment has not been developed commercially in the United States.

An electrically operated concrete surface-type vibrator with a frequency of 3,600 vibrations per min, operating on an 11-in. by 48-in. platform, and a pneumatically operated internal concrete vibrator of $2\frac{3}{4}$ in. diameter and 6,000 rpm were tried out for the cohesionless material at Franklin Falls.

Although these units were believed to be indicative of those attainable with practical vibration units especially designed for operations on a large scale, the results of tests on material *A* were inferior to those obtained by rolling equipment. In addition, honeycombing caused by trapped water, resulting in water voids up to the size of walnuts, was discovered.

These tests were made on material *A* which was of relatively low permeability. Similar tests on material of high permeability might prove more satisfactory, provided (as would be necessary for success) sufficient water could be supplied to keep the material saturated.

(*d*) *Compacting Cohesive Soils.*—Crawler tractor-drawn sheepsfoot rollers have now become standard equipment for compacting cohesive soils. The objections to smooth rollers are the same as those described for cohesionless soils. Data on the use of grooved rollers for cohesive soils are very limited. Tampers, except for limited areas (as previously described for cohesionless soils), have not been used in the United States.

(*e*) *Moisture Content.*—Coarse, cohesionless materials require the most water for optimum compaction. Coarse, cohesive soils require the minimum water content, and the quantity of required water again increases as the soil approaches a clay.

Cohesionless Soils.—The maximum possible amount of water which will permit satisfactory operation of compacting equipment is necessary for cohesionless soils. In fact, when the materials are very coarse and free draining, it is impracticable to retain in the voids sufficient water to obtain maximum compaction.

Cohesive Soils.—Economical compaction of cohesive soils is definitely tied to accuracy in supplying the correct amount of moisture, and frequent field tests are required. Too much moisture is better than too little for several reasons: (1) Evaporation tends to reduce the moisture as the rolling progresses; (2) an excessive amount of moisture is impossible, since the equipment will bog down; and (3) the greater this moisture content, the less will be the tendency of later expansion due to saturation as described in paragraph (*g*), under heading, "2. Factors Affecting Control Density of Soils."

(*f*) *Thickness of Layers.*—The correct thickness of layers for all soils is a function of the equipment used; the nature of the material; and, in cohesionless materials, the permeability. The proper thickness should be determined from a test embankment. Within limits, there is a relation between thickness of layers and the number of passes required for a given density. When too thick, the density may vary considerably throughout the layer. All thicknesses of layers are specified to be measured after compaction.

Cohesionless soils can be placed as deep as 18 in. in thickness, but 12 in. is usual. Cohesive soils are usually placed in 6-in. layers, although 9-in. layers are permissible with certain materials and proper equipment.

(*g*) *Passes of Rollers.*—As mentioned previously, the effect of the tractor on compaction has less effect proportionally on cohesive than cohesionless soils. However, it is obvious that the tractor has some effect on compaction of all classes of material. Test embankments are frequently compacted by two or three rollers drawn by a tractor, and from such tests the number of passes are specified. Therefore, the use of more than that number of rollers with a single tractor will require more passes, for the same degree of compaction, than indicated in the test fill.

Modern practice permits the contractor to draw multiple rollers up to twelve with one tractor. The greater number of passes necessary to get the desired density are frequently paid for as an extra. Therefore, it would seem desirable to determine, in the test fill, the relative compacting effect of the roller and to specify either a limit to the number of rollers to be drawn by a single tractor or to increase the number of passes with increase in the number of rollers without extra compensation.

For cohesionless materials, the tractor has the preponderating effect on compaction. Therefore, the number of passes should be defined as the number of passes of the tractor treads. For cohesive materials, the number of passes is measured by passes of the rollers.

Three tractor passes on cohesionless materials and six roller passes on cohesive materials are about the minimum. When the required passes exceed about seven to fourteen, respectively, the use of a heavier roller or thinner layers will probably prove more economical.

(*h*) *Mixing Materials.*—It is seldom necessary to mix two available materials for the purpose of obtaining a combined mixture of a desired gradation in the construction of a dam, since materials of different gradation can be most economically used in dam construction. However, mixing is sometimes required for other purposes when:

(1) Certain cohesive materials containing too much water for successful compaction are mixed with relatively dry materials, or

(2) When broken rock not suitable for rock fill is to be mixed with soil.

The first item, or mixing to adjust the water content, is not necessarily mixing in the true sense of the word, since the materials have only to be placed in juxtaposition in order for some water, during rolling, to pass from the wet to the dry soil. This may be accomplished successfully when the wet layer underlies the dry layer in the borrow pit and a single scoop of the shovel encounters some of each; or the materials may be placed in alternate layers on the dam.

Shattered shales, from necessary excavations, not suitable by themselves in dam construction, have been mixed successfully with suitable soil in dam construction. Extensive tests on this procedure have been conducted by the Pittsburgh, Pa., District of the Army Engineers.

It has been found that a mixture of two parts of soil to one part of shale will be safe if properly handled. The mixing is accomplished by depositing windrows of each material and mixing with a bulldozer. Successful mixing is accomplished when the shale particles are completely embedded with no cavities for the shales to occupy if they should disintegrate.

(*i*) *Compacting Foundations.*—No method has yet been devised for compacting foundations composed of cohesive materials. The use of dynamite is the only method successfully used for compacting cohesionless foundations. At the request of this Sub-Committee, A. K. B. Lyman,[3] M. Am. Soc. C. E., has written a paper on this subject.

4. Gagings for Dams

(*a*) *Settlement Gages.*—The purpose of settlement gages in dams is: (1) To check theories of settlement; and (2) to determine if a fine saturated soil in the dam and in the foundation is settling as fast, and hence is attaining strength as fast, as anticipated.

Fig. 4 shows two types of vertical settlement gages. Type *A* contemplates the use of a small pipe set in a vertical position in the earth fill, or its foundation, and welded at the bottom to large, spreading base-plates resting on a sand pocket in the fill or in the foundation material. Each base-plate is shown slotted so that the next pipe can pass through it and thus bunch the several pipes from different levels more closely than they could be bunched if the base-plates were

[3] "Compaction of Cohesionless Foundation Soils by Means of Explosives," by A. K. B. Lyman, *Proceedings*, Am. Soc. C. E. (publication pending).

not slotted. Brackets are shown welded to the pipe and base-plate to furnish stiffnesss at the joint. Three or four such brackets for each connection would probably be desirable. On the other hand, it might be still more desirable if the entire base-plate were made of a ribbed casting in order to spread the resistance better over a considerable area and furnish greater stiffness of the base-plate than could be had with plate steel.

With a sufficient spread of the base-plate and using flush joint pipe it is probable that the resistance of the base-plate to vertical movement through the material of the fill would be sufficient to overcome the friction of the soil with the vertical pipe and cause the pipe to respond accurately to the settlement

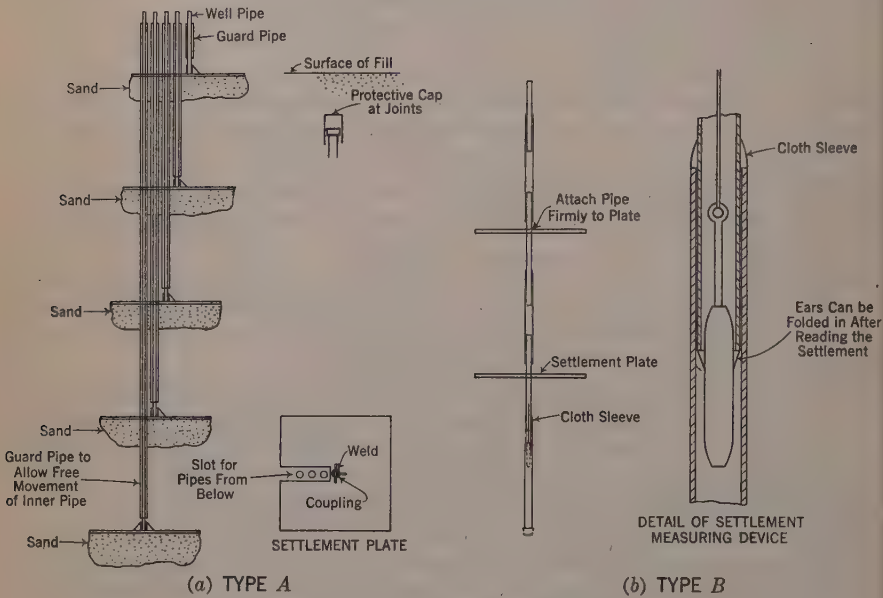


Fig. 4.—Vertical Settlement Gages

at the elevation of the base-plate. However, if there is any question about this movement, then each pipe should be surrounded by a guard pipe to allow free movement to the inner pipe with reference to the soil and thus eliminate the friction.

By means of this apparatus, and with the pipes protruding above the fill with the cap provided with a small hole to permit equalization of air pressure at all times, it would be possible to measure settlement by keeping track of the elevations of the tops of each pipe and to measure water level of hydrostatic level at each elevation by sounding the water elevation in the pipe. In a very tight soil with a small yield of water it is possible that evaporation from the pipe would somewhat obscure the indication, but with a large sand pocket and large base-plate and relatively small pipe it is believed that the yield would be enough in almost any material to reduce the errors due to evaporation to a very small, if not negligible, amount.

The installation of such pipe in the earth fill as it builds upward: (1) Would require putting a barrel around the group of pipes extending above the surface of the fill and routing the fill equipment, graders, rollers, etc., around this spot, resorting to hand tamping immediately adjacent to the pipe where the roller could not function; or (2) it would involve digging a pit down each time to the buried tops of the pipes, putting on a short extension which would not protrude above the surface of the fill, capping the pipe, and backtamping so that the fill could proceed again for, say, a 5-ft lift above the ends of pipe, at which time they would be uncovered again and extension made. An inverted cap *M* (Fig. 4(*a*)) could be put over the end of each pipe to permit rolling across the fill above the pipe without putting a load on the end of the pipe. At each time of such uncovering the elevation of the top of each pipe would be measured before and after putting on the extension so that a progressive record of settlement during construction and at each elevation would be obtained.

Gage *B*, shown in Fig. 4(*b*), is in use by the U. S. Bureau of Reclamation. The plates are welded to sections of pipe which are installed with sleeve joints to adjacent pipes so that the entire apparatus can telescope as required to accommodate itself to the settlement. Settlement is measured by a device in the form of a plumb-bob with two ears which is lowered into the pipe and drawn up until the ears engage the bottom of each section of pipe. Elevation is then read, after which a wire will fold in the ears and permit raising the device out of the hole. This arrangement offers the advantage that no guard pipe need be provided, and only one vertical pipe assembly is needed instead of several corresponding to each level at which the settlement is to be determined.

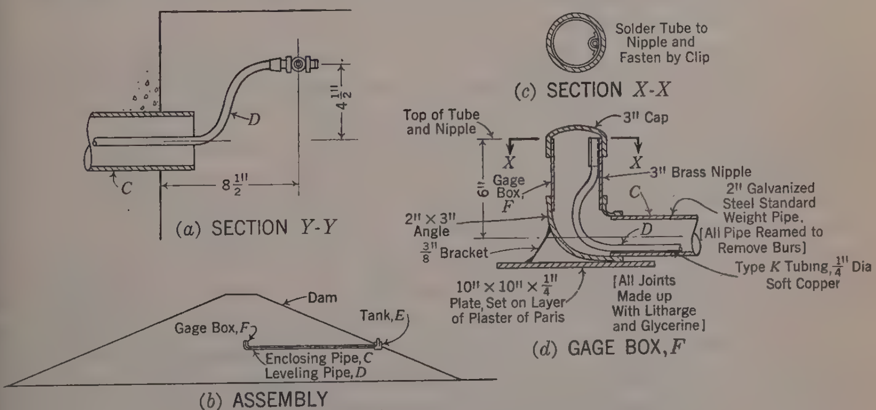


FIG. 5.—DETAILS OF TYPE *C* SETTLEMENT GAGE

Gage *C*, shown in Fig. 5, has been suggested to avoid the objectionable feature of having to install vertical gage pipes in the earth fill as it builds upward. The idea for this gage was originally suggested by E. B. Smith.[4] It contemplates the use of nearly horizontal pipe leading from the point of measurement to the outside of the fill instead of the use of vertical pipe. The horizontal pipe can be buried in a trench and carried to an observation station out-

[4] *Bulletin No. 112*, Iowa Eng. Experiment Station, Iowa State College, Ames, Iowa.

side of the fill and at the level required, after which it constitutes no nuisance or obstruction to the future fill operations.

The apparatus consists of a 2-in. enclosing or drainpipe, C, extending into the fill as shown in the assembly sketch of Fig. 5 at such an angle that, for any anticipated settlement, it will always drain to the outside of the dam. Threaded into the drainpipe, C, is a $\frac{1}{4}$-in. leveling pipe, D, which extends from a tank, E, outside of the dam, to a gage box, F. This pipe is curved at each end to take up expansion and contraction.

To determine the settlement of the dam at the elevation of the gage box it is necessary to fill the leveling pipe, D, and tank, E, with water until it overflows into and out of the drainpipe, C. The level of the water in the tank, E, will then be the same as that at the upturned end of the leveling pipe, D, in the gage box. The original elevation of the latter being known, the settlement can be determined easily.

In order that the drainpipe will drain after total settlement of the dam has occurred, it is recommended that it be given an initial slope such that it will be level after an extremely liberal estimate of differential settlement at each end has taken place. Should the drainpipe, C, crack, due to unequal settlement or heavy rolling, it will still drain and the leveling pipe should still remain intact. The drainpipe must be large enough to drain when only partly full because, should it fill, a siphonic action will develop and defeat the purpose of the test. Assuming the initial slope of the drainpipe is 3 ft, Fig. 5 indicates that the tank would be about 5 ft 8 in. high. Assuming the leveling pipe to be empty, the cock, B (Fig. 6), under the tank, E, should be closed and the tank filled with water. This will require 2 gal of water.

After the tank is filled, the cock is opened full and the water allowed to fill the leveling pipe. This procedure will force out any air which may be entrapped in a slight summit. The tank will hold more than enough water to fill the leveling pipe, and the surplus will spill into and run out of the drainpipe. When the surges in the tank have ceased, it is necessary to add more water slowly until the addition of a slight amount will no longer result in a rise of water surface. The large diameter of the tank, compared with that of the leveling pipe, will result in very slight surges, but such as do occur can be reduced by cracking the cock, B. Should the leveling pipe be full, when a reading is desired, the same procedure should be followed to force out any air which has accumulated. In cold climates the gage may be drained after each measurement through the plug, G. However, kerosene or antifreeze mixtures can be used.

Gages of this type have been installed at Tionesta, Pa.; Kingsley, Nebr.; Pinopolis Dam, South Carolina; San Gabriel No. 1, California; and Caddoa, Colo. It has been used also in a number of highway fills in Iowa. Reports of its use have been very satisfactory.

Fig. 5 shows an installation for only one gage. Obviously, by extending the drainpipe and using additional leveling pipes, additional gages at about the same level may be installed. The drainpipe may be used also to contain connections to hydrostatic-pressure or earth-pressure gages if desired.

(*b*) *Water-Pressure Gages.*—Gages for measuring the hydrostatic pressure of water are for the purpose of checking flow nets and also to determine if any

detrimental hydrostatic pressures exist in the dam or the foundation. They are of two types—piezometers and direct-pressure gages.

The piezometer type consists simply of a vertical pipe, set in the dam, the lower end being open to allow pore water to enter. The elevation of the water in the pipe measures the potential of pore pressure at its lower end.

Sometimes the pipe is perforated throughout its length. In this case the measured potential is the average between that at the lower end and that at the phreatic line. This will usually be less than that at the phreatic line. The Sub-Committee sees no object in using perforated pipe.

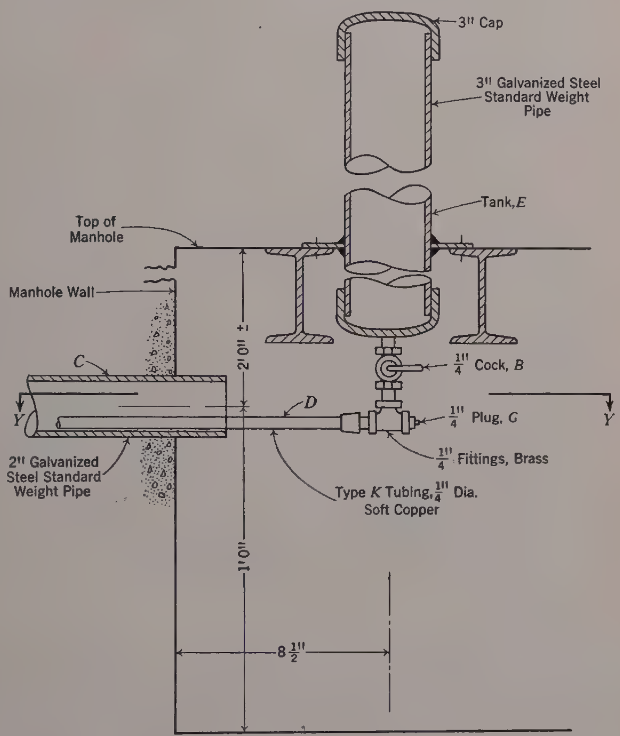


FIG. 6.—TANK *E*, FIG. 5(*b*)

The use of such piezometers is usually for the determination only of the elevation of the phreatic line but sometimes to determine pore pressures at various places as, for instance, when separating pore pressures from total pressures obtained by total pressure gages as described subsequently in paragraph (*c*), under heading "4. Gagings for Dams."

When used to determine the phreatic line, the lower end of the piezometer should be only slightly below that line, since, if it is below, it will give readings which are too low by an approximate amount:

$$e = d \sin^2 \phi \qquad (1)$$

in which, as in Fig. 7, d = the depth of lower end of piezometer below the phreatic line; and ϕ = the angle of the phreatic line with the horizontal.

The lower end of the piezometer pipe should be encased in a pocket of sand to assist in collecting or discharging water due to fluctuations in piezometer readings. Piezometers may be combined with gage A, as shown in Fig. 4(a), but not with gage B, as shown in Fig. 4(b), since the latter is not watertight and is subject to the same objections as previously described for perforated pipe piezometers.

A piezometer pipe can be carried horizontally from any point in the dam to the outside, and from thence be connected to a piezometer running up the face

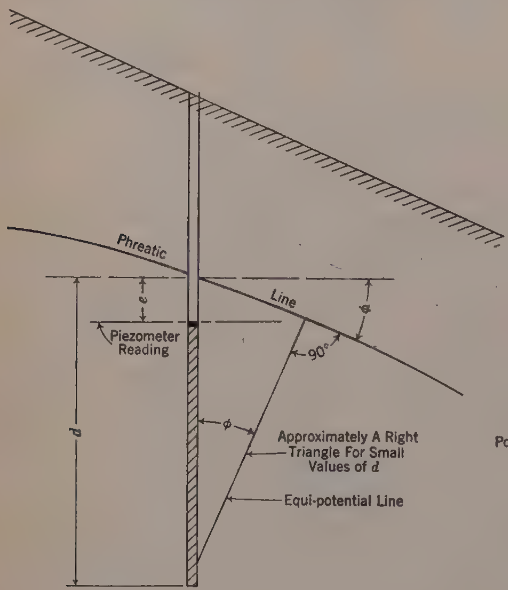


FIG. 7

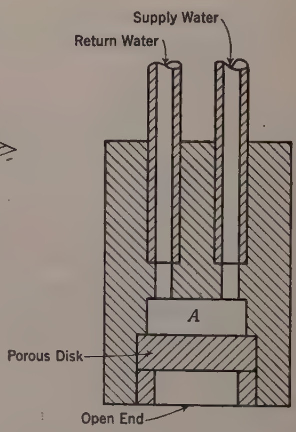


FIG. 8

of the dam or to a pressure gage. The horizontal pipe in the dam could be carried in the enclosing pipe of gage C, shown in Fig. 5. Single-pipe horizontal piezometers are likely to entrap air and thereby invalidate readings. Double pipes that can be flushed out are preferable, as described subsequently.

Piezometers are quite satisfactory in pervious materials and are most frequently used in such cases. However, for very impervious materials, it takes too long for the piezometers to adjust themselves to changes in pressure. Even with the lower end terminating in a sand pocket, as previously mentioned, a material change in pore pressure may take days or weeks to manifest itself in a piezometer.

Of course, there is the same objection to vertical piezometers as there is to vertical settlement gages, as previously described, if installed during the construction of the dam. However, it is possible to install them later by jetting in sandy material or borings in cohesive material. The difficulty for cohesive

material is that, as explained heretofore, the desirable sand pocket at the foot of the pipe would be missing. For these reasons, except for very pervious materials, a positive pressure indicator, which does not require a flow of water to the gage in order to register, is desirable.

A gage of this kind[5] has been used, consisting of a small chamber within the dam in which there is a flexible diaphragm, on one side of which is pore water and on the other side pressure water supplied through a pipe from a gage on the outside of the dam. The gage registers the pressure required to balance the pore pressure and move the diaphragm slightly. Such movement is indicated electrically through a conduit which also is used as the pipe to supply the pressure water. However, electrical troubles have caused the abandonment of this type of gage and the U. S. Reclamation Service has devised the type indicated in Fig. 8. Pore water enters the open end, passes through the porous disk, and enters the chamber A. The supply and return pipes lead to the outside of the dam. They are kept constantly full of water and are connected to a Bourdon gage which measures the pore pressure. Two pipes are used so that the water in the pipes can be circulated to remove accumulations of air which would cause errors of measurement.

Manufacturers of Bourdon gages will guarantee $\frac{1}{2}\%$ to 1% accuracy for standard instruments and as low as $\frac{1}{6}$ of 1% for instruments which are specially calibrated for small variations in head.

(c) *Soil Pressure Gages.*—The Sub-Committee has been unable to contact any one who has used successfully any type of soil pressure gage when installed in firm soil and entirely surrounded by soil. This is attributed to the arching effect of the earth where applied to the measuring diaphragm used in gages which have been tried out. Fair results have been obtained from such gages when they have been attached flush with concrete walls embedded in the soil.

Where installed in semi-liquid soils, as for the installations in the cores of the Miami dams, this arching effect is not noticeable, and reliable readings have been obtained. The usual type of gage used for this purpose is the Goldbeck Cell.[6]

Soil pressure gages of the usual type register both soil and water pressure. Therefore, if the soil pressure alone is desired, a water pressure gage must be installed adjacent to the point in question and its reading be subtracted.

Respectfully submitted:

Glennon Gilboy
Francisco Gomez-Perez
R. M. Haines
J. H. Stratton
K. B. Woods
W. P. Creager, *Chairman*

July 24, 1940

[5] See *Engineering News-Record*, August 17, 1939, p. 52.

[6] *Soil Mechanics Bulletin*, U. S. Waterways Experiment Station, September 1, 1939, p. 13.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GENERAL WEDGE THEORY OF EARTH PRESSURE

Discussion

By Karl Terzaghi, M. Am. Soc. C. E.

Karl Terzaghi,[27] M. Am. Soc. C. E. (by letter).[27a]—Mr. Peckworth suggests that the paper would not be complete without discussion and experiments proving or disproving three statements concerning cohesive earth, designated A, B, and C. Since Mr. Peckworth admits himself that there is no such thing as an angle of repose of cohesive earth, a discussion of statement A appears to be unnecessary. Statement B does not refer to a controversial subject. Every one of the existing theories of earth pressure leads to the conclusion expressed by this statement, and the writer does not know of any observations which are at variance with this conclusion.

The observations described by Mr. Peckworth in connection with his statement C seem to indicate that the break in the ground behind a vertical bank with a height h always occurs at a distance $\frac{h}{2}$ from the rim of the bank. This statement agrees with the writer's experiences. All the breaks of this category which are known to the writer from personal experience have one important feature in common. All of them started with the formation of a tension crack at a distance $\frac{h}{2}$ from the rim of the bank. Once this crack had developed the bank was permanently weakened and sooner or later it failed along a curved surface of rupture which connected, in some manner, the bottom of the crack with the foot of the bank. This observation led the writer to the tentative conclusion that the tensile stresses on the horizontal surface next to the upper rim of a vertical bank in cohesive earth are a maximum at a distance $\frac{h}{2}$ from the rim of the bank. In order to determine whether this rule is also valid for perfectly elastic materials, the writer made model tests several years ago, in

Note.—This paper by Karl Terzaghi, M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Howard F. Peckworth, M. Am. Soc. C. E.; February, 1940, by Jacob Feld, M. Am. Soc. C. E.; April, 1940, by M. G. Spangler, Assoc. M. Am. Soc. C. E., and June, 1940, by Donald M. Burmister, Assoc. M. Am. Soc. C. E.

[27] Dr. Ing., Visiting Lecturer, Harvard Univ., Cambridge, Mass.

[27a] Received by the Secretary September 4, 1940.

his Vienna (Austria) laboratory, with a vertical bank made of gelatin and determined the tensile stresses along the upper horizontal surface from the results of strain measurements. The results are shown in Fig. 12. The dotted line represents the deformation of the bank due to its own weight, and the ordinates of the plain curve with reference to the section through the upper horizontal surface represent the horizontal tensile stresses σ_h along this surface. These stresses are a maximum at a distance of about $\frac{h}{2}$ from the rim of the bank. Hence, if a vertical bank of gelatin fails, it fails in the same manner as a bank in cemented sawdust, hard rock, or stiff clay. The mechanical properties of the bank material merely influence, to some extent, the shape of the surface of rupture between the bottom of the tension crack and the foot of the bank.

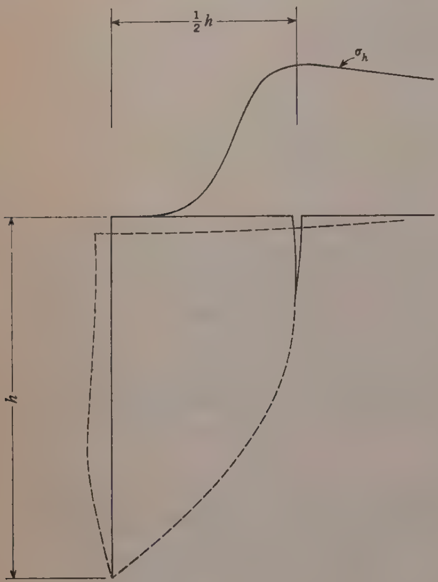


FIG. 12

The preceding conclusions lose their validity for soils without any cohesion, such as clean sands. Fig. 3 shows that the shape of the surface of sliding in a mass of sand behind the vertical sides of a timbered cut is very similar to the surface of rupture in a cohesive material, such as that shown in Fig. 12. For an angle of internal friction, $\phi = 31°$, and an angle of wall friction, $\delta = 26°$, the general wedge theory for cohesionless materials leads even to the conclusion that the surface of sliding in the sand intersects the horizontal surface at a distance equal to one half the depth of the cut. This surface of sliding is practically identical with the surface of rupture, shown in Fig. 12; yet all these similarities are purely accidental and have no significance. For values of ϕ greater than 31° the distance between the upper rim of the cut and the upper edge of the surface should be smaller than $\frac{h}{2}$. Since the angle of internal friction of sands is very seldom smaller than 31°, the width of the top of the sliding wedge adjoining the sheeted sides of cuts in sand should, as a rule, be smaller than $\frac{h}{2}$. The experiences of the writer confirm this conclusion. In some cases the writer found that the measurable settlements of the surface next to cuts in sand did not extend beyond a distance of 0.4 h.

At the end of his discussion Mr. Peckworth requests the definition of several of the terms which are used in connection with earth pressure. The writer suggests the following:

> "Active earth pressure" is the pressure exerted by the earth on a yielding lateral support at the instant when the earth fails by sliding in a downward direction along an inclined surface of sliding.

In sandy soils a very small lateral yield of the support is normally sufficient to reduce the lateral pressure of the earth to a value close to the active earth pressure. Therefore, the designer is usually justified in assuming that the lateral pressure of a sandy soil on a slightly yielding lateral support is approximately equal to the active earth pressure. However the yield required to produce actual shear failure in the soil can be many times greater than the yield required to approach closely the active earth pressure.

> "Passive earth pressure" is the greatest lateral pressure that can be exerted on a soil by a structure such as an arch bridge abutment or the lower part of a sheet-pile bulkhead.

The mobilization of the passive earth pressure is associated with a lateral compression of the soil. Even in sandy soils the lateral compression required to increase the lateral earth pressure to a value close to the passive earth pressure is very considerable. Hence, if the stability of a structure depends on the lateral resistance of the earth, one should never count on more than one third, or at the most one half, of the computed value of the passive earth pressure.

> The "angle of repose" is equal to the angle between the horizontal and the slope of a heap of soil produced by dumping the soil from some elevation.

For perfectly clean and dry sand or gravel the angle of repose is fairly independent of the height of the heap and the method of dumping, and it is approximately equal to the angle of internal friction of the sand in the loosest state. The angle of repose of moist sand and of cohesive soils depends essentially on the height of the heap and on the method of dumping. Hence, in connection with such soils, the angle of repose has no meaning.

The "angle made by material pushed off the slope from the top" depends on the degree of disintegration prior to the process of pushing and on the weather at the time of pushing. In the rainy season the material may flow off without being pushed. Hence, no definite rules can be established.

The term "angle made by the line of rupture and the face of a sheeted trench" has no definite meaning because this angle increases from a maximum value at the toe of the bank to zero degrees at the upper surface, as shown in Fig. 3. The angle at the toe of the bank depends on both the angle of shearing resistance ϕ and the angle of side friction. If these two values are known, the slope angle of the surface of sliding at the toe of the bank can be determined with close approximation by means of Mohr's diagram.

Referring to the results of the measurements of the pressure in a subway cut in Berlin, illustrated by Figs. 4 to 9, Mr. Feld expressed the opinion that the results of field measurements of lateral pressure in back of sheathing are too indefinite to be of any value because "Slight changes in the bracing as installed change the reactions entirely." It will be shown in the following that the scattering of the measured strut pressures from the average is by no means as important as the appearance of the curves in Fig. 9 would seem to indicate.

Nevertheless, no matter how important that scattering may be, every one of the struts must be made strong enough to stand the greatest pressure that may act on it under the most unfavorable field conditions, because the failure of a single strut is likely to cause a failure of the entire bank support. Therefore, every designer has a vital interest in securing information on the greatest possible deviations from the statistical average, and this information can be secured only by pressure measurements in full-sized cuts such as those described in the paper.

Regarding the results of the measurements, Mr. Feld merely commented on the scattering of the empirical pressure distribution from the statistical average. However, from a practical point of view, the designer is chiefly interested in the deviation of the measured values from those theoretical values on which the design is based. Therefore it may be more interesting to compare the measured values with those obtained by theory.

In 1936 the writer proposed,[9] on the basis of theoretical considerations, that the triangular pressure distribution should be replaced by the pressure distribution represented by the trapezoidal area *A B C D* in Fig. 9. The size of this area is equal to 1.12 times the theoretical pressure P_{an} determined by Equation (1). The factor 1.12 represents a margin for the inevitable scattering of real pressures from the statistical average. In order to compare the errors associated with the traditional and the proposed method of computing the pressures in the struts, assume that the soldier beams are cut immediately below point *D* in Fig. 13 and hinged at the elevation of the struts Nos. 2 to 4.

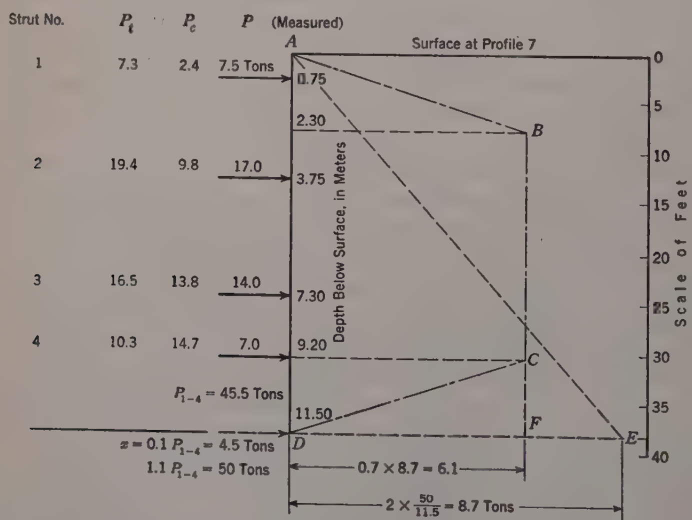


FIG. 13

At point *D* they are freely supported. The unknown horizontal pressure on the support at *D* is assumed to be equal to 10% of the sum P_{1-4} of the measured pressures on the struts 1 to 4. Hence, the total lateral pressure on the sheath-

[9] "Distribution of the Lateral Pressure of Sand on the Timbering of Cuts," by Karl Terzaghi, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Vol. I, Cambridge, Mass., 1936.

ing is equal to 1.1 P_{1-4} for each profile. The values of the pressures on the individual struts and the data concerning the elevation of the struts with reference to the surface of the ground have been published.[10] For profile 7 these data are given in Fig. 13. At the site of this profile the cut has a depth of 11.5 m (37 ft 9 in.). The measured strut pressures are listed in the figure under the heading "P." The sum P_{1-4} of these pressures is 45.5 tons. (In this profile the pressure in the highest strut has not been measured. In the other profiles it ranged between 6.5 and 8.5 tons. Hence, it is assumed that the pressure in the highest strut is equal to 7.5 tons.) Thus, the total lateral pressure on the sides of the cut adjoining profile 7 is equal to 1.1 times 45.5 = 50 tons. If this pressure acts like a hydrostatic pressure, the triangle $A\ E\ D$, Fig. 13, representing a total pressure of 50 tons, is obtained as a pressure area. The strut pressures produced by this load are listed in the figure under the heading "P_c." On the other hand, if the load is assumed distributed over the sides of the cut as indicated by the trapezoidal area $A\ B\ C\ D$, representing a total pressure of 1.12 times 50 = 56 tons, the resulting pressures are as listed in Fig. 13 under the heading "P_t." The three columns of numerical values show at a glance that the errors associated with the traditional method of computation represented by the column "P_c" are intolerable. For the highest strut the measured pressure is more than three times greater than the computed one. On the other hand, the agreement between the measured values and the computed values P_t is very satisfactory.

A similar computation has been made for profiles 4 and 10 which represent the most abnormal profiles in a set of seven. Table 1 contains the ratio between the measured strut pressures P and the values P_t computed by means of the trapezoid method. The table shows, first of all, that the errors associated with the method of computation proposed by the writer are much smaller than those involved in the traditional method of computation (see values of P_c in Fig. 13). Second, it shows that the highest ratio between the measured and calculated values for the strut pressures is considerably smaller than the lowest customary factor of safety. Hence, a low factor of safety seems to be sufficient for taking care of the influence of unequal wedging and other accidental features of the construction operations on the pressure in the struts. However, the method of computation proposed by the writer involves a radical departure from accepted practice, and no conscientious engineer can be expected to change

TABLE 1.—RATIOS OF $\dfrac{\text{MEASURED STRUT PRESSURE}}{\text{COMPUTED STRUT PRESSURE}}$

Strut No. (see Fig. 13)	Profile Nos.:		
	4	7	10
1	1.19	1.03	1.17
2	0.60	0.88	1.06
3	0.99	0.85	0.91
4	0.77	0.68	0.42

[10] "Mitteilung über die Messung der Kräfte in einer Baugrubenaussteifung," by A. Spilker, *Die Bautechnik*, 1937, Heft 1.

his methods without ample empirical evidence that the proposed modification is justified. This evidence can be obtained only by pressure measurements in full-sized cuts. Since the paper was published in *Proceedings* (October, 1939), such measurements have been made in one cut in Boston, Mass., and in four cuts in Chicago, Ill., with a depth from 30 to 50 ft. Although all these cuts were made in clay, the distribution of the lateral pressure on the sheathing is strikingly similar to that shown in Fig. 9 of the paper, for a cut in clean sand.

In the last paragraphs of his discussion Mr. Feld commented on the writer's disregard of the reaction taken by the soil from the embedded portion of the soldier beams supporting the sheathing. His objections are shared by Professor Spangler. Both discussers seem to believe that the high position of the centroid of the lateral pressure illustrated by Fig. 9 is chiefly due to the influence of the neglected reaction on the distribution of the lateral pressure. According to this conception, the real distribution of the lateral pressure should be very different from that shown in the figure and much more similar to the hydrostatic distribution.

The writer ignored the lower reaction because he believes that its influence on the distribution of the lateral pressure is too small to be of any practical consequence. The reasoning which led him to this opinion is illustrated by Fig. 14, which is a section through the cut at profile 7. The measurements in

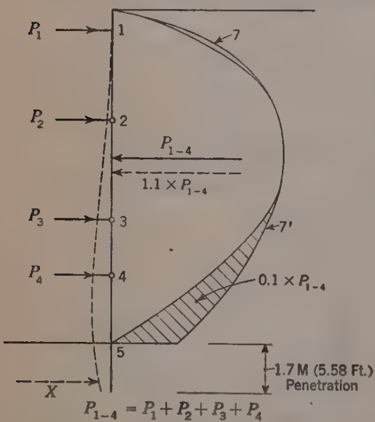


FIG. 14

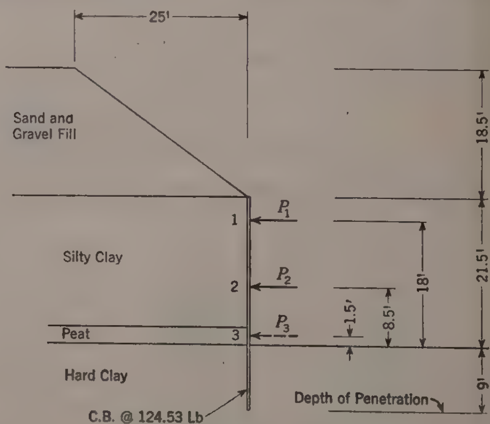


FIG. 15

the field furnished the values for the pressure on the struts at points 1 to 4. They were equal to 7.5, 17.0, 14.0, and 7.0 metric tons (16,500, 37,600, 31,000, and 15,400 lb), respectively. In order to estimate the corresponding distribution of the lateral pressure, the writer assumed that the soldier beams are hinged at points 2, 3, and 4, and are freely supported at points 1 to 5. Then he determined by trial the outer boundary of a pressure area which satisfies the condition that the pressure is in equilibrium with the measured reactions at the points 1 to 4. The result of this operation is shown in curve 7, Fig. 14.

In reality the soldier beams are continuous and they are bent as indicated by the broken line in Fig. 14. In order to produce a deflection of that type

one must add to the strut reactions P_1 to P_4 a soil reaction X. At the same time one must shift the curved boundary of the pressure area from its original position 7 into a position 7′ in such a manner that the pressure represented by the shaded area shown in Fig. 14 is equal to the reaction X. The pressure curve 7′ thus obtained resembles the theoretical pressure curves computed by the writer[9] in 1936. The shaded area shown in Fig. 14 represents 10% of the measured pressures P_1 to P_4 on the struts 1 to 4. The soldier beams in the cut in Berlin consisted of I-beams, 12 in. wide, which were driven to a depth of not more than 5.5 ft below the bottom of the cut. Therefore, the writer believes that the effect of the neglected soil reaction X on the distribution of the lateral pressure should be appreciably smaller than that shown in Fig. 14.

In order to prove or to disprove this opinion, it would be necessary to cut the soldier beams at point 5 and to measure the effect of this operation on the strut pressures. No such operation has been performed in Berlin. However, quite recently, A. Casagrande, Assoc. M. Am. Soc. C. E., has made a similar test on a cut in Boston, because the conditions were such that the soil reaction X was likely to be exceptionally important. Fig. 15 is a section through the cut. The soldier beams were much heavier than those used in Berlin and, although the depth of the sheeted part of the cut was only 21.5 ft, the beams were driven to a depth of 9 ft below grade into hard clay. To a depth of 18.5 ft the cut was made with sloping sides, as shown. The remainder of the cut was made with vertical sides. The lagging consisted of horizontal boards resting on the inner side of the flanges of the soldier beams. At points 1 and 2, Fig. 15, the soldier beams were supported by struts. At point 3 a strut was loosely fitted to the soldier beam without carrying any pressure. After the pressures P_1 and P_2 on the struts 1 and 2 had been measured, the vertical soldier beam was cut immediately below point 3 and the pressure on the struts was measured once more. In Table 2 the symbols P_1 to P_3 represent the pres-

TABLE 2.—Pressure on Struts 1 to 3 Before and After Soldier Beams Were Cut

Profile No.	P_1	P_1'	P_2	P_2'	P_3	P_3'	n	n'	$\frac{P_1' + P_2' + P_3'}{P_1 + P_2 + P_3}$	$\frac{n'}{n}$
1	11.8	12.2	10.8	9.9	0	4.6	0.76	0.65	1.154	0.85
2	13.0	12.5	15.3	15.4	0	5.1	0.73	0.62	1.145	0.85

sure on the struts 1 to 3, in tons, before the soldier beams were cut, and n is the ratio between the elevation of the centroid of the pressures on the struts above the bottom of the cut and the depth h of the sheathed part of the cut. The symbols P_1' to P_3' and n' represent the corresponding values after cutting. The effect of the cutting on the total measured lateral pressure, and on the elevation of the centroid of the measured pressures, is given in the last two columns. These values show that the soil reaction X was approximately equal to 15% of the total lateral pressure, and that the elimination of this reaction lowered the centroid of the pressure by about 15%. In order to compare the

conditions for the end support of the soldier beams in the cuts in Berlin and the cuts in Boston, respectively, the following data may be studied:

Item	Description	Berlin	Boston
1	Total depth of penetration, in feet	5.5	9.0
2	Ratio, $\frac{\text{height of sheathed sides}}{\text{total depth of penetration}}$	7.2	2.4
3	Weight of soldier beams, in pounds per foot	50.0	53.0
4	Width of soldier beams, in inches	12.0	12.0
5	Width of flanges, in inches	6.0	12.0
6	Area of contact between soil and the outer face of the flange in the buried part of the soldier beam	2.8	9.0

These values show that the lower ends of the soldier beams in Berlin were very much less confined than those in Boston, and they were much more flexible. In Boston the soil reaction X was about equal to 15% of the total lateral pressure. Hence, the writer feels justified in assuming that the influence of the neglected soil reaction X in Berlin on the intensity and the distribution of the lateral earth pressure was much less important than the influence indicated by the shaded area in Fig. 14. In order to eliminate, entirely, the error due to disregarding the soil reaction X, future measurements in cuts with soldier beams should be combined with measurements of the effect of cutting the soldier beams on the pressure in the struts. The author dares to predict, however, that the influence of the cutting on the pressures in the struts will usually be very small. This forecast agrees not only with the preceding analysis but also with the following field observation: In one of the cuts in Chicago the lower ends of the soldier beams were located above the bottom of the cut. Nevertheless, the centroid of the measured strut pressures was close to the midheight of the sides of the cut, and the value of the ratio between the elevation of the centroid and the depth of the cut is located within the range of scattering of the corresponding value for the other cuts, with soldier beams whose lower ends were driven 5 ft into the ground.

The method of computation represented by the trapezoidal area $A\ B\ C\ D$ in Fig. 13 can be used if the system of bracing includes soldier beams with their lower ends embedded in the ground. If no such soldier beams are used, it is advisable to replace the trapezoidal area $A\ B\ C\ D$ in Fig. 13 by the area $A\ B\ F\ D$. The area $C\ D\ F$ represents a substitute for the shaded area in Fig. 14.

In his discussion Professor Burmister presents interesting suggestions regarding the methods of determining the angle of shearing resistance, the distribution of the soil reactions over the base of retaining walls, and the effect of drainage on the earth pressure. However, all these topics are beyond the scope of the paper.

In conclusion the writer wishes to express his gratitude to the discussers for their stimulating comments.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CHANNELIZATION OF MOTOR TRAFFIC

Discussion

BY GUY KELCEY, M. AM. SOC. C. E.

GUY KELCEY,[50] M. AM. SOC. C. E. (by letter).[50a]—To all who made substantial contributions in presenting this most interesting subject, both in its original preparation and in the constructive discussion that followed, the writer wishes to express his appreciation. Some of the questions raised have been explained in a supplementary paper.[51]

Mr. Waters suggests that two weak points in the theory of channelization are: (1) It assumes that all motorists are familiar with the arrangements at each crossing; and (2) it largely ignores the pedestrian. The writer feels that proper channelization should simplify crossing problems to the point where "what to do next" is obvious. If a motorist is in doubt, at any time, the channelization treatment is not a good one or should not have been attempted until other conditions, which prevent a proper treatment, have been corrected.

Channelization is not in itself a cure-all. It often solves the problem of the pedestrian by eliminating trouble-causing factors. Its primary purpose is to solve the problem of the vehicle; thereafter, greatest simplicity and certainty of vehicle movement having been attained, further steps to solve the pedestrian problem, if needed, become simpler and easier. If, then, channelization does not solve the difficulty in certain instances, it paves the way for the best and simplest solutions of pedestrian and other remaining problems.

Mr. Crosby asks if there should not have been two more islands shown in Fig. 19. Islands in two opposite throats of a right-angled intersection usually prevent troublesome irregular vehicular movements within an intersection. If pedestrians are numerous the two islands are often best installed in the two opposite intersection throats which carry the greatest pedestrian load. If pedestrian flow is heavy in all directions, islands may be placed in all four

NOTE.—This paper by Guy Kelcey, M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Messrs. W. L. Waters, C. J. Tilden, and T. M. Matson; April, 1940, by W. W. Crosby, M. Am. Soc. C. E.; May, 1940, by Messrs. Julian Montgomery, R. M. Reindollar, Irving Mack, Bruce D. Greenshields, T. W. Forbes, and James S. Bixby; and June, 1940, by George H. Herrold, Hawley S. Simpson, Arthur G. Straetz, Burton W. Marsh, and Virden A. Rittgers.

[50] Signal Service Corporation, Elizabeth, N. J.

[50a] Received by the Secretary September 12, 1940.

[51] "Traffic Channelization Methods," by Guy Kelcey, *Civil Engineering*, October, 1940, p. 645.

throats for their protection and for more complete vehicular control. If no pedestrians are involved, islands are usually installed in opposite throats of the roadway carrying the least traffic unless there is need, as is often the case, to divide the main flow.

Mr. Crosby also questions if, in Figs. 31, 32, 33, 35, 36, and 37, provision should have been made for left turns. Roadways, which intersect at acute angles, usually have cross roadways behind the intersection, in each direction, which can be used and which eliminate the need for the left-turn provision. However, the solutions shown in Figs. 31, 33, 35 (if enough roadway space permits), and 37 may be adjusted to permit left turns. On the other hand, it is often desirable, on the grounds of expediting and safeguarding the greatest number, to prohibit or block out the limited demand for left turns at intersections of this kind.

Particular reference is made to the discussion of Professor Forbes who has made a new and most valuable contribution in his very clear suggestions for measuring psychological and physiological values.

Discussions by Messrs. Tilden, Matson, Reindollar, Montgomery, Mack, Greenshields, Bixby, Straetz, Simpson, Herrold, Marsh, and Rittgers have all added constructively and substantially to the subject.

Corrections for *Transactions: Proceedings*, December, 1939, page 1650, sentence beginning line 26 should read "In fact a study of the accident experience involving a large representative group of drivers in Connecticut (particularly accident repeaters), for a period of six years, discloses that only 1.3% of all drivers are accident repeaters and that 80.9% are normally efficient"; page 1652, line 7, after "the car," insert "the road"; page 1657, sentence beginning line 13 should end " * * * to make the turn without interfering with other lanes of traffic"; and page 1673, end of first paragraph under "Acknowledgments" add "The writer acknowledges particularly, and with sincere appreciation, many valuable suggestions, ideas, and criticisms from the following: Miller McClintock and Maxwell Halsey, Yale Bureau for Street Traffic Research; Robert A. Mitchell, traffic engineer of Philadelphia; E. V. Miller, Assoc. M. Am. Soc. C. E., design engineer of Arizona Highway Department; Lacey Murrow, Assoc, M. Am. Soc. C. E., director of highways of the State of Washington; Harry E. Neal, traffic engineer of the Ohio Highway Department; W. F. Rosenwald, traffic engineer, Minnesota Highway Department; the late George Schlesinger, M. Am. Soc. C. E., National Paving Brick Association; T. J. Seburn, traffic engineer of Kansas City; Leslie Sorenson, traffic engineer, City of Chicago; and many others."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SEALING THE LAGOON LINING AT TREASURE ISLAND WITH SALT

Discussion

BY JOHN W. PRITCHETT, ASSOC. M. AM. SOC. C. E.

JOHN W. PRITCHETT,[11] Assoc. M. Am. Soc. C. E. (by letter).[11a]—The wide dispersion of clay, its availability in large quantities, and the low cost of obtaining and handling it account for its increase in use and importance as a material for construction. With increased research and increased knowledge in the modes of testing and utilizing this material, there is being developed scientific knowledge upon which more definite conclusions may be reached as to the behavior of clays of various types and under various usages. The interesting paper by Mr. Lee is a distinct contribution to such knowledge. It brings to attention the importance of thorough analyses of clay materials proposed to be used in construction where imperviousness is important, and in localities where such types of clays are likely to be encountered. Whether or not such types are of common occurrence in a given locality is a matter that should receive the attention of those concerned with designing structures having clay as part of the structure or the foundation.

The practicability of using clay for the purpose of obtaining impervious lining for salt-water storage pits in part of the East Texas oil field has recently become a matter of importance. An area of about 100 sq miles in the southwestern part of this large oil field lies within the Neches watershed. On November 1, 1939, it was estimated that 1,567 oil wells in this field and within the Neches River watershed were producing salt water. The daily allowable production of oil in this area is 20 bbl per well. An estimate of salt water being produced in 1940 was 74,000 bbl per day. The maximum per well has been from 900 to 1,000 bbl per day. It has been estimated that at least 31 wells were each producing 450 bbl or more per day. Analysis of the water being

NOTE.—This paper by Charles H. Lee, M. Am. Soc. C. E., was published in February, 1940, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1940, by Messrs. G. B. Bodman, and John D. Watson.

[11] Member, State Board of Water Engrs., Austin, Tex.

[11a] Received by the Secretary August 1, 1940.

produced from a representative well in this area is as follows (in parts per million):

Total solids	61,170.0	Iron	7.0
pH	8.2	Manganese, less than	0.02
Phosphorus alkalinity	0.0	Sodium	0.0
Total alkalinity	435.0	Carbonate	0.0
Silica residue	220.0	Bicarbonate	531.0
Turbidity	55.0	Sulfate	643.0
Fixed salts	58,630.0	Chlorides	35,500.0
Calcium	1,240.0	Fluoride	1.8
Magnesium	572.0	Nitrate, less than	0.4
Total hardness	5,443.0	Selenium	0.0
Sodium (calcium)	1,036.0		

The problem was to formulate some plan for the operation of oil wells in this area in such a manner that salt-water contamination of the Neches River could be abated sufficiently so that the quality would meet the requirements for municipal, industrial, and rice irrigation uses in the lower reaches of the stream, and would also not be harmful to fish life. As a part of this larger problem, tests were made on clay and asphalt samples which are interesting in so far as they apply to the results reported by Mr. Lee.

TABLE 11.—RESULTS OF SOIL ANALYSIS

Description	Sample No.:													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14
(a) Physical Characteristics														
Cylinder No.	..	1	2	..	3	..	..	6	7	..	..	10	11	12
Density, wet, pounds per cubic foot	..	126	126	..	122	..	..	119	122	..	..	122	118	116
Density, dry, pounds per cubic foot	..	104	104	..	100	..	..	91	96	..	..	96	93	89
Percentage of water in mix	..	20	22	..	22	..	..	30	26	..	..	26	26	30
Soil classification [a]	S	..	C_l	S	C_s	S	S	C_l	C	S	S	C	C	C
(b) Mechanical Analysis														
Colloids (%)	..	..	18	..	17	..	..	17	16	..	..	25	35	35
Clay and colloids (%)	..	..	28	..	28	..	..	27	35	..	..	46	85	57
Silt (%)	6	..	42	10	52	5	5	23	43	5	7	42	7	36
Fine sand (%)	85	..	25	27	19	75	17	38	17	50	57	10	8	6
Coarse sand (%)	9	..	5	13	1	20	78	12	5	45	36	2	0	1
Specific gravity	..	..	2.71	..	2.66	..	..	2.66	2.70	..	..	2.68	2.72	2.74
Porosity (%)	..	..	39.5	..	32.8	..	..	45.0	42.7	..	..	42.4	45.1	47.8
Voids ratio	..	..	0.652	..	0.488	..	..	0.817	0.747	..	..	0.736	0.820	0.914

[a] S = sand; C = clay; C_l = clay loams; and C_s = silty clay.

Asphalt Sample.—A sand-asphalt mixture pressed into a cylinder about 4 in. in diameter and 2 in. thick was submerged in a solution containing 40,000 ppm chlorides. This material is similar to that used for earth stabilizer on asphalt highway construction. After being submerged ten days or two weeks, only

TABLE 12.—Tests of Salt Treatment; Samples Submerged in Tap Water

No.[a]	Description		Condition After Submergence		
	Color	Surface	Two hours	Ten hours	Twenty hours
			(a) Mixed with City Water		
2	Light chocolate	Smooth	Almost disintegrated	Disintegrated	Disintegrated
3	Light red	Lightly checked	Almost disintegrated	Disintegrated	Disintegrated
5	Cream	Smooth	Good condition	Disintegrated	Disintegrated
8	Light tan	Smooth	Lower half disintegrated [b]	Disintegrated	Disintegrated
9	Medium red	Smooth	Disintegrated	Disintegrated	Disintegrated
12	Grayish yellow	Smooth	Disintegrated	Disintegrated	Disintegrated
13	Red	Lightly checked	Lower half disintegrated [b]	Disintegrated	Disintegrated
14	Red	Smooth	Disintegrated	Disintegrated	Disintegrated
			(b) Mixed with City Water Containing 10,000 PPM of Chlorides		
2	Darker chocolate	Granular	Almost disintegrated	Disintegrated	Disintegrated
3	Light red	Smooth	Slightly disintegrated, lower surface	Two thirds disintegrated [d]	One half disintegrated [d]
5	Darker cream	Smooth	Lower third slowly disintegrating	Three fourths disintegrated [d]	Disintegrated
8	Darker tan	Smooth	Lower surface slowly disintegrating	One half disintegrated [e]	One half disintegrated [d]
9	Medium red [c]	Smooth	Almost disintegrated	Disintegrated	Disintegrated
12	Darker yellow	Smooth	Upper fourth intact, but critical	Disintegrated	Disintegrated
13	Darker red	Lightly checked	Lower two thirds disintegrated	One third disintegrated [f]	One half disintegrated [e]
14	Darker red	Smooth	Lower two thirds disintegrated	One half disintegrated [f]	One half disintegrated [h]
			(c) Mixed with City Water Containing 20,000 PPM of Chlorides		
2	Darker chocolate	More granular	Almost disintegrated	Disintegrated	Disintegrated
3	Darker red	Granular	Lower surface, continued disintegrating	One half disintegrated [d]	Two thirds disintegrated [d]
5	Darker cream	Granular	Intact; good condition	One half disintegrated [d]	Disintegrated
8	Dark tan [c]	Smooth [g]	Lower surface, continued disintegrating	Two thirds disintegrated [d]	Two thirds disintegrated [d]
9	Dark red	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
12	Dark yellow [c]	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
13	Darker red	Lightly checked [g]	Little disintegration; top stable	One third disintegrated [h]	Two thirds disintegrated [h]
14	Darker red	Smooth [g]	Lower two thirds disintegrated	One half disintegrated [d]	Disintegrated
			(d) Mixed with City Water Containing 40,000 PPM of Chlorides		
2	Darker chocolate	Smooth	Disintegrated	Disintegrated	Disintegrated
3	Much darker red	Smooth	Firm; good condition	One third disintegrated [e]	Two thirds disintegrated [h]
5	Darker cream	Smooth [g]	Firm; excellent condition	One half disintegrated [d]	Almost disintegrated [d]
8	Much darker tan	Smooth [g]	Lower surface, continued disintegrating	Two thirds disintegrated [d]	Two thirds disintegrated [d]
9	Much darker red	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
12	Much darker yellow	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
13	Much darker red	Smooth [g]	Firm; good condition	One half disintegrated; cracked	Two thirds disintegrated [h]
14	Darker red	Smooth [g]	Firm; lower surface disintegrating	One third disintegrated [h]	Disintegrated

[a] Samples air dried 3.5 days. [b] Remainder firm. [c] No change in color. [d] Top shaped, but critical. [e] Top shaped and firm. [f] Top cracked; soft and critical. [g] Salt spots. [h] Top shaped; some stability.

the slightest indication of deterioration was observed. The sample was then set aside, allowing the water to evaporate. After three months the sample was examined. It was found to be encrusted in a layer of salt crystals, but plainly showed cracks, indicating deterioration. On breaking away the salt crust, almost complete breakdown of the material was observed. The outer edges fell apart easily, and any part of the sample could be mashed and crumbled between the fingers.

Clay Sample.—For purposes of examination and test 14 samples of clay were secured. These samples were considered representative of the available clays in the southern part of the area where proposed storage pits would probably be constructed. The results of the soil analyses were as shown in Table 11.

Of the 14 samples secured, eight were mixed to putty-like consistency with (*a*) city tap water, (*b*) water with 10,000 ppm of chlorides, (*c*) water with 20,000 ppm of chlorides, and (*d*) water with 40,000 ppm of chlorides. Each mixture was rolled by hand into 2-in. balls and allowed to dry in air for 3.5 days, the purpose being to duplicate the tests made by the author, as described under the heading "Tests of Salt Treatment." One full set of each mixture was submerged to two thirds the diameter in tap water for observation. Table 12 shows the results of these tests. Four of the samples were duplicated as per mix (*d*) (water with 40,000 ppm of chlorides), and after 3.5 days of air drying, were similarly submerged, but in water having 40,000 ppm of chlorides. Table 13 shows the results of this test.

In contrast with the results obtained by Mr. Lee, after submergence for two hours, the samples mixed with salt water showed nearly the same degree of disintegration as did the samples mixed with fresh water. After ten hours submergence most of the samples were either disintegrated or nearly so, whereas Mr. Lee states that his No. 3 and No. 4 showed no change in condition after two hours submergence.

In addition to the foregoing tests, 70 g of each of the eight clay samples shown in Table 13 were well broken down after being air dried and then covered

TABLE 13.—Tests of Salt Treatment; Samples Submerged in Salt Water

Sample No.[a]	Condition After Submergence for:		
	Two hours	Ten hours	Twenty hours
2	Disintegrated	Disintegrated	Disintegrated
3	One half disintegrated [b]	Disintegrated	Disintegrated
9	One third disintegrated [b]	Three fourths disintegrated [d]	Four fifths disintegrated [b]
13	Slowly disintegrating [c]	One half disintegrated [e]	Two thirds disintegrated [e]

[a] Samples air dried for 3.5 days. [b] Top shaped, but critical. [c] Top firm. [d] Top shaped, but plastic. [e] Top shaped; some stability.

to a depth of one-half inch with water containing 40,000 ppm of chlorides. Three and one-half days later the surplus water was drained off and the samples were allowed to dry in the air to a putty-like consistency, at which stage none of them appeared to have lost any noticeable degree of cohesion. This material

was then rolled into balls and allowed to dry. After three months the balls were hard like baked clay, with no cracks or signs of deterioration.

It was desirable to determine whether or not the clay samples, if used as lining for salt-water storage pits, would lose their cohesive properties if submerged continuously in water having a chloride content as high as that produced from the oil wells. The results of the investigations indicated that under proper methods of construction, several of the clays analyzed might be expected to produce a tank lining sufficiently impervious for all practical purposes. Tests on such samples taken from cylinders prepared in the laboratory, moulded under a pressure of 100 lb per sq in., showed a permeability of less than 0.025 or 1/40 of an inch per day per foot of head per foot of thickness. Chemical analyses of the soluble contents of the clay samples were not obtained.

Many storage tanks previously constructed in this area were found to be losing considerable water through seepage. Newer tanks have clay linings, with observation wells at each tank. Inspections will be made to determine whether or not excessive seepage losses occurred.

The problem of providing an impervious lining for pits used for the storage of salt water is somewhat different from the problem presented by Mr. Lee where salt treatment was used to obtain a more impervious lining for a reservoir for the storage of fresh water. The lining of the salt water storage pits would not be subjected to fresh water treatment after the introduction of salt water. Just what effect the continuous exposure to salt water will have on the permeability of the lining is a matter of much interest. Mr. Lee's paper brings to attention the importance of giving due consideration to the effect of salt on clay materials proposed for construction uses.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

AXIOMS IN ROADWAY SOIL MECHANICS

Discussion

By A. A. Eremin, Assoc. M. Am. Soc. C. E.

A. A. Eremin,[7] Assoc. M. Am. Soc. C. E. (by letter).[7a]—In the form of axioms the author has shown various fundamentals of influence of the subgrade soil behavior on a roadway pavement. The axioms would be considerably clarified if their relation to the natural laws of soil mechanics were explained. The object of soil mechanics study, as in any other science, is to develop natural laws and find their limitations.

It is well known that the relation of pressure to the moisture content in soils is somewhat similar to the relation of stress to strain in solid materials. With this conclusion in mind, the author's axioms relating to the influence of the fluctuation in moisture content on deformation of highway slabs may be clarified. Movement and fluctuation of the moisture content in soils follow the hydrodynamic laws. Therefore, the author's axioms relating to displacements and deformations in a saturated soil lying on the impervious sloping strata are obvious.

It is almost impossible to classify soils according to their percentage of sand or clay content. However, familiarity with the characteristic features of sandy and clay soils may help to make interpretations of behavior of soils with various content of sand and clay.

Further development of soil mechanics applied to highway construction will be based on the natural laws of physics and mechanics. Summary of the axioms without clearly determined princeps may be lengthy and difficult for practical application.

Note.—This paper by Henry C. Porter, M. Am. Soc. C. E., was published in February, 1940, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1940, by Victor J. Brown, Esq.

[7] Associate Bridge Engr., Bridge Dept., Div. of Highways, State Dept. of Public Works, Sacramento Calif.

[7a] Received by the Secretary August 7, 1940.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF HINGES AND ARTICULATIONS IN REINFORCED CONCRETE STRUCTURES

Discussion

BY A. A. EREMIN, ASSOC. M. AM. SOC. C. E.

A. A. EREMIN,[3] Assoc. Am. Soc. C. E. (by letter).[3a]—Interesting information concerning Mesnager and Considère hinges is contained in this paper. However, some problems of hinge design still remain to be clarified. It would be useful to know the assumptions on which the equations for computation of stresses in the hinges cited by the author were based.

Evidently Eq. 1 can be applied only in the computation of the stresses within the elastic limit. Nevertheless, the author used Eq. 1 for computing the ultimate stresses in his diagram in Fig. 2. Furthermore, Eq. 1 was developed with the assumption that steel bars in the hinge form a truss and that the concrete part of the hinge was considered as a rigid member. Actually, the concrete, even if it is reinforced with steel hoops or transverse wiring, is elastic. This is a serious limitation of Eq. 1, and it suggests some need for simplifying the equation by omitting the minor factors.

Mr. Mesnager has limited the bar slenderness to a maximum ratio of $l/r = 40$ and has considered the concrete cover as a means of protecting the steel bars from corrosion. If slender steel bars in the hinge were used and their strengths varied with the stiffening effect of the concrete cover as shown in Figs. 3 and 4, then that hinge would lose the characteristic features of the Mesnager hinge and its design would be based on an entirely different principle.

The advantage of the compressive concrete members reinforced with spiral reinforcement is in the increased strength of the concrete core formed by the spiral reinforcement. Eqs. 7 to 11 however were developed for a rectangular section considering concrete outside spiral reinforcement as a part of the section. At high stresses the concrete cover in the Considère hinge is generally cracked and peeled off. The highest efficiency of the Considère hinges was obtained when concrete cover was omitted. In the temporary Considère

NOTE.—This paper by George C. Ernst, Assoc. M. Am. Soc. C. E., was published in April, 1940, *Proceedings*.

[3] Associate Bridge Engr., Bridge Dept., Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

[3a] Received by the Secretary August 19, 1940.

hinges used for construction purposes the concrete cover outside the spiral reinforcement is not required. In the finished structures the temporary Considère hinges are generally cast in the permanent concrete members. The Considère temporary hinges, without concrete cover, are especially convenient in the construction of reinforced concrete arch bridges.

Corrections for *Transactions:* April *Proceedings*, page 591, in the caption for Fig. 2 change "f_3" to "f_y"; page 592, line 28, change "P" to "T"; page 595, line 10, after "in cross section" add "at the hinge"; page 597, line 2, after "contact faces," add "During casting, sheet steel forms were used to maintain the roller radius, as shown in Fig. 6(a)"; page 597, line 4, change "4,940" to "4,750"; in Fig. 7 the horizontal grids are 5,000, 10,000, and 15,000 pounds per linear inch; and page 601, line 9 following Eq. 11, change "342 r" to "350 r."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MASONRY DAMS

A SYMPOSIUM

Discussion

By Messrs. F. A. Nickell, Leslie W. Stocker, Barton M. Jones, P. E. Gisiger, Joseph A. Kitts, S. O. Harper, and R. F. Blanks

F. A. Nickell,[75] Esq. (by letter).[75a]—Even in application to engineering structures geology is a broad field. Mr. Crosby has done a remarkably thorough job in outlining the more common geological problems in the selection and investigation of dam sites. As a general thing different types of rocks have certain outstanding characteristics so that principles can be defined and applied to sites where these rocks occur. The subject does not lend itself readily to simple classification of conditions, nor to formal treatment, but examples are of considerable value since they reflect experience.

Each dam site presents problems in a way different from those previously encountered. General concepts about foundations are difficult to apply in solution of specific cases, and there is danger in drawing a close analogy between sites having apparently identical geology. In his introductory statement, Mr. Crosby points out that the success of a dam depends on many factors. It is necessary to know the significance of features in the foundation as a basis for correct design. Although the nature of formations is important, the critical response in rocks due to influences coming from the dam is commonly the effect of small adjustments within members. This merely emphasizes that minor features, such as seams, may determine the ultimate success or failure of the structure.

The weight of the dam and thrust from reservoir storage creates conditions of concentrated stress that the rocks have not experienced before, perhaps even prior to erosion of the canyon. The resulting deformation suffered by the foundation is in accordance with the net physical character of the component

Note.—This Symposium was published in May, 1940, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: September, 1940, by Messrs. William P. Creager, J. R. Shank, George R. Rich, Robert A. Sutherland, Ross M. Riegel, Paul Baumann, W. A. Perkins, L. J. Mensch, and Lewis H. Tuthill.

[75] Head Geologist, Bureau of Reclamation, Denver, Colo.

[75a] Received by the Secretary August 29, 1940.

rocks, and thus may be largely elastic or purely a matter of consolidation. Although the extent of probable adjustment can be computed with reasonable accuracy from a knowledge of materials in the foundation and is, of course, extremely important to the designer of dams, continued submergence may cause a decrease in available support through combined effects of saturation and removal of soluble components. Mr. Crosby's point in regard to employment of an experienced geologist during investigation and early construction is, therefore, well considered.

There are two paramount objectives in all construction: First, to build permanently, and second, to obtain efficient operation. It should be recognized that these objectives are modified according to local conditions and purpose of each project, but security of the dam should never be in doubt. What constitutes efficient operation is subject, as Mr. Crosby indicates, to the location and general requirements for storage. A leaky dam site may permit adequate regulation of floods, provided seepage is noninjurious, but a similar loss would be unduly costly to an irrigation project in an arid region. Furthermore, the significance of certain geological conditions may be widely different at the dam site and in the reservoir area. Reservoir seepage in certain cases is permissible, but the same amount of leakage might be dangerous around the dam where the percolation path is short.

The stability of foundations is contingent upon bearing power, shearing strength, and resistance to sliding, under the assumption that the concerned rocks retain their original character—in short, will not soften, be dissolved, or lose constituents by piping. These factors can be computed in the laboratory and, provided the samples have been carefully selected, the values so determined for compression, shearing strength, and frictional resistance are ordinarily sufficient for design of most dams. How representative the measurements are depends on whether unknown defective or weaker members exist in the foundation. It follows, therefore, that exploration of the dam site should be so exhaustive that no reasonable chance exists of overlooking critical features that may cause failure. This points again to the need for the expert knowledge of the geologist, and, moreover, one familiar with engineering.

Geological Investigation.—The value of exploratory results depends alike on thoroughness of investigation and accuracy of interpretation. Mr. Crosby has laid considerable emphasis on the need for careful geological appraisal so that any discussion of exploratory methods based on another's experience involves some repetition.

Dam sites are chosen by aerial, topographic, and geological comparison. Investigation is a matter of securing adequate information for design of all suitable types of dams. The diamond drill is used extensively in both preliminary and final exploration. It is desirable to have a hole at least 2 in. in diameter for maximum core return. Recovery of representative material alone is not enough. Usually percolation tests with packers or casing are required to segregate leaky horizons where grouting or other protection is needed. In preliminary studies the configuration of the water table is important since it reflects the course of natural ground-water movement. The elevation of the water table is determined by recession to a static level or

ascension following bailing. There is a trend toward large borings with shot drills allowing periscopic and visual inspection of the hole. Few sites for high dams are explored satisfactorily without tunnels, shafts, and trenches at several levels.

The interconnection of open cracks through the rock may have great importance. Leaks discovered by packers and other devices can be traced with coloring fluids, diagnostic liquids, or electrolytes. Some unusual methods have been devised by enterprising investigators. Free passage to the river along openings in lava at a proposed site in Oregon was shown by appearance in the stream of kerosene dumped into a neighboring drill hole. The spread of joints and solution channels at the Alcova Dam site, in Wyoming, was vividly indicated during investigation by a burst of compressed air into an angle hole passing under the river.

Mr. Crosby has mentioned the use of geophysical methods in dam investigation. The possibilities are not generally appreciated and the interest of geophysicists should be aroused in engineering and foundation problems. The identification of porous media by the electrical resistivity method has not been utilized in analogy of seepage through permeable foundations. The interpretation of seismograph records, based on elastic properties of rocks, might disclose a relation between seismic response and deflection under high dams. The period of foundations and resonance of structures during earthquakes are matters of vital concern in seismically active areas and might be established in the same manner.

The percolation path in permeable layers is closely allied to the design of the so-called "floating" structures and with uplift under other types of dams. In many instances the slope can be portrayed by drawdown measured in small holes around a large central well used for pumping.

The writer has been impressed by the desirability of having field tests to check experimental values determined in the laboratory and to eliminate the element of personal judgment so far as possible. Experiments made with ease in the laboratory can be conducted only with difficulty, if at all, on rock in place; yet the added strength due to confinement affords an undetermined factor of safety that might be utilized for closer design if conditions could be positively analyzed. Definite progress in this direction is seen on the part of many construction agencies. The compressive strength of sediments at the Table Mountain site, in northern California, and of granite at the Twin Springs site, in Idaho, has been calibrated by loaded column and hydraulic jack with rather uniform results. An elaborate program of bearing tests is being conducted along zones of weakness as well as in normal rock during excavation and preparation of the foundation for Shasta Dam, in California.

Foundations of Typical Rocks.—The trend in design is an expression of accumulated experience with earlier dams. Because the response of materials in concrete dams can be governed fairly well by strict specifications, foundations remain the most questionable element in design. A reasonable effort in exploration and instrumental measurement will repay in ultimate benefits many times its initial cost.

Naturally, the problems vary according to the composition of foundations. In the introductory remarks the writer indicated that generalization is usually unwarranted, but that examples ordinarily are instructive. Mr. Crosby has maintained this idea in presenting illustrations of dams on different types of foundation materials. Most large concrete dams—for example, those built by the Bureau of Reclamation—are constructed on foundations of hard rock. Hard rock may be taken to include granite and other bodies of deep-seated origin, as well as certain sedimentary, metamorphic, and volcanic formations, which, considered as a group, offer foundations generally acceptable for any type of dam. Although hard rocks are considered best suited to high dams, they have common weaknesses, including crushed zones, joints, contacts, irregular weathering effects, and laminations of foliate rocks. The importance of most defects diminishes with depth. An empirical rule holds that excavation should be carried to levels where support is adequate and at which the rock below can be treated satisfactorily—for example, by grouting and drainage.

Dams on Granitic Rock.—Mr. Crosby has described treatment of faults in foundations of granite and the remedy is more or less orthodox for inactive fault zones. Crushed zones in the foundation for Seminoe Dam on the North Platte River, in Wyoming, were excavated of all soft material and refilled with concrete. A less serious feature exposed in the floor at Grand Coulee Dam, in the State of Washington, was excavated some distance below the average level of the adjacent rock. Bartlett Dam on the Verde River, in Arizona, is a multiple arch, 286 ft high, on a foundation chiefly of rather fine-grained granite. Strong joints parallel the canyon, forming a series of vertical rock slabs on the right abutment where buckling might occur without sufficient excavation and adequate lateral support.

Formations in some regions have characteristic defects. The Boise Mountains in Idaho, largely of granite, contain numerous broad zones of minutely sheeted and friable rock. This condition would have a considerable effect on excavation for a dam—for example, at the Twin Springs site on the Boise River.

The common problems in preparation of foundations of granitic rocks are well defined and resolve largely into questions as to depth of excavation and requirements for grouting and drainage.

Dams on Metamorphic Rocks.—Foundations of metamorphic rocks may be entirely acceptable but have an element of added danger in foliate structure. One of the contributing causes in the failure of the St. Francis Dam, in California, as Mr. Crosby indicates, was the schistosity paralleling the left abutment face. Weathering in banded rocks is usually selective so that decay persists irregularly below the average level of unaltered material. Since sliding is the main hazard, added resistance may be given the foundation, as was done at the Easton Dam on the Yakima River, in the State of Washington, by providing a saw-tooth outline.

Dams on Volcanic Rock.—Mr. Crosby's few comments on dams and reservoirs in volcanic rocks barely touch upon the innumerable problems associated with formations of volcanic origin, which, as a group, are unpredictable. Brecciated contacts, contraction joints formed during cooling, lava tunnels, porous

intravolcanic sediments, and a depressed water table are ordinary difficult features. Volcanic rocks occur over vast areas, including a large part of the Northwest and future demand for storage in those regions will continue to raise perplexing geological problems. Reasonably effective storage can be obtained where the lava in the reservoir area has a blanket of sediment, as is the case at the American Falls, Minidoka (Idaho), and Wickiup (Oregon) dam sites.

Without protective cover, impervious beds in an alternating series of flows and sediment may limit losses. A secure dam can be built usually with provision only against uplift and piping, if the lava foundation offers adequate support. Various remedial measures have been successful, including grouting, concrete cutoff walls in the permeable layers, and blanketing. It is evident that a satisfactory dam in volcanic rocks may become ineffective and possibly unsafe if the reservoir is raised and storage increased.

Dams on Sedimentary Rocks.—Sedimentary rocks common in every section of the United States are in the foundation of a large number of dams. Conditions of deposition and later modifying effects tend to produce variations in character even among similar kinds, and the illustrations offered by Mr. Crosby can represent only a few of the problems connected with them. Formations of the same geological classification vary widely in physical ability according to texture, bedding, degree of cementation, sequence of strata, and internal structure, so that, irrespective of the geological age of the concerned rocks, the situation at each site has to be analyzed separately.

Sandstone with great frictional resistance internally and between layers is generally well qualified for moderate to heavy loads. Limestone and certain chemical equivalents may be highly suitable on one hand or dangerous otherwise, depending on geological processes experienced by these relatively soluble rocks. With added clay content, complications also increase. The acceptability of these sediments is crudely expressed by a graph with sandstone, limestone, and clay at the apexes of a triangle, representing, respectively, "best," "good," and "fair" quality stone.

In the future, more dams (many of concrete on account of spillway and other requirements) will be built on rather soft foundations. There will be a need at these sites for field investigations on bearing ability such as are described by Mr. Crosby and which have been made in detail at the Conchas Dam in New Mexico and for the Possum Kingdom Dam in Texas.

Dams on Pervious Foundations.—Mr. Crosby neglected to discuss masonry structures of restricted height on pervious foundations. Dams across streams with deep alluvial cover on bedrock are occasionally necessary. Since it is not practical to found them on solid rock, and because spillway requirements are usually exorbitant, the dam is designed as an overflow structure, secure from uplift and sliding, and obtaining support by means of a reinforced-concrete matte of adequate length, in cases resting on a forest of piles. The Imperial Dam on the Colorado River, in Arizona and California, involves every aspect of this kind of problem—a deep, pervious fill and a vigorous stream. The design was made on the basis of laboratory tests of representative material to determine percolation path, sliding resistance, internal friction, and bearing. Geology has little to offer directly in this kind of situation, but should be valuable in

analysis of material, studies of retrogression, and development of stream characteristics.

Summary.—A conservative principle in construction is the adaptation of design to the foundation. Because reasons other than geological limitations govern the selection in many cases, the best plan is reduction of unit load to within safe limits. Exhaustion in the number of unused good sites, the extreme conditions introduced by very high dams, and the tendency to build where necessary regardless of the geological complications accentuate the demand for adequate knowledge and ways of obtaining it. The paper by Mr. Crosby gives a broad cross section of common problems connected with different types of foundations and is suggestive of the things to look for during investigation. An opportunity for study is offered by each new dam. Besides thorough field tests to corroborate laboratory experiments, a valuable source of information will be realized in observations subsequent to construction. Since concepts of foundations are speculative as to the long-time effects of load and submergence, continued measurements are desirable during operation following completion of the dam. For this purpose various instruments are available, including pressure cells, stress meters, strain meters, and tiltmeters embedded in rock beneath and downstream from the dam to indicate distribution of stress, bulging or differential adjustment, excessive uplift, and other pertinent data. It is encouraging to note the installation of instruments at some dams in reflection of a growing appreciation of foundation problems.

Leslie W. Stocker,[76] M. Am. Soc. C. E. (by letter).[76a]—In setting forth general considerations governing the arrangement and details of construction joints in dams, and in presenting facts, and deductions by himself and others from such facts, as to joint treatment in existing dams and the behavior of some dams after construction, Mr. Steele has rendered a service for which the designer of a dam should be very grateful.

A 5-ft height of lift has been common for many years, doubtless originating with practical construction considerations, and, having been found to give satisfactory results, in general, has become practically standard. The spacing of transverse joints has been decreasing from the time, not a great many years ago, when they were first introduced, to the distance of about 50 ft which now prevails. Data in Table 11 indicate that approximately 10 to 20 years ago a number of designers thought favorably of the practice of using a very wide spacing in the lower part of a gravity dam, halving it above, and in some cases quartering it at a still higher level. This appears rather venturesome, unless a long time elapses between the times of pouring the concrete placed above and below the elevations at which the spacing changes, as there must be considerable tendency for the formation of a crack as a downward extension of the partial joint. It would be interesting to know the reasoning behind the use, in the upper part of the Cignana Dam, of the partial joint extending only part way from the upstream face; this seems a direct invitation for a crack to form as a continuation of the joint through the downstream concrete.

[76] Senior Civ. Engr., Public Utilities Comm., Bureau of Eng., San Francisco, Calif.

[76a] Received by the Secretary August 9, 1940.

In the case of the enlarged O'Shaughnessy Dam of the City of San Francisco (dam No. 60 in Table 11), the construction joints (see Fig. 25) had to be so treated as to perform the duty (unusual in high dam construction) of transmitting stresses from the original section into a large addition in such a way that on completion the addition should bear its proper share of the load, the work of enlargement being done while the original dam was in service under varying load conditions. So far as that function is concerned, the principal construction joint is the longitudinal one between the old and the new work. The design of the joint along the original downstream face is of particular interest.

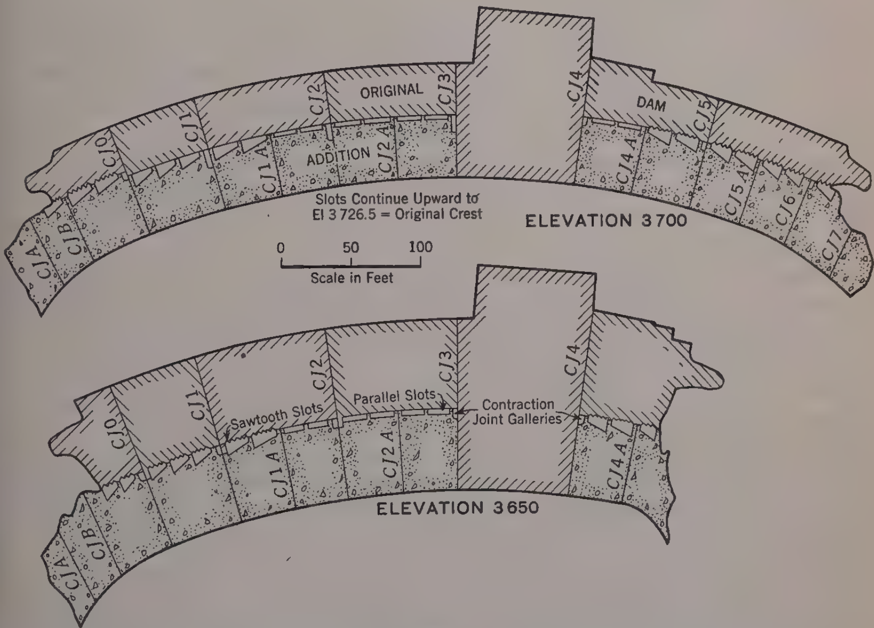


Fig. 25.—Horizontal Sections of O'Shaughnessy Dam as Enlarged, Showing Arrangement of Slots Along Longitudinal Joint, and of Radial Contraction Joints, 150 Ft and 200 Ft Above Base of Addition

O'Shaughnessy Dam is of the curved gravity type, having a radius of 700 ft measured to the upper (vertical) part of the upstream face. Its height is 312 ft above stream bed, or 430 ft above the lowest point in the foundation. It was originally planned for its present height, but on account of economic considerations was initially built only to 226.5 ft above stream bed, leaving 85.5 ft for future addition. This initial work was completed in 1923. The base section, below stream-bed level, was built to the full thickness required for the ultimate structure, to avoid future cofferdam work. A block containing six outlet valves and their discharge conduits was built to full thickness for its entire height. The remainder of the original dam had only the thickness necessary to suit its own height, leaving a level concrete surface 80 ft wide at the stream-bed elevation to serve as foundation for the future addition. The downstream face was formed in 5-ft steps corresponding to the lifts of concrete. The radial

construction joints of the original work are spaced, in general, 97 ft apart at the upstream face. The south end block, 130 ft long, cracked transversely 85 ft south of the southerly joint; nothing indicates that the spacing is too great elsewhere. The radial joints were not grouted. Sheet copper water stops of the U-type were placed across these joints near the upstream face; a longitudinal copper water stop was provided along the crest, with its lower half embedded and the upper half folded down and enclosed in a wood-covered recess, to be unfolded and become embedded in the new concrete of the later addition. A water stop similarly embedded and housed was set around the perimeter of each of the 18 siphon spillway ports.

Concrete construction of the addition, to raise the crest 85.5 ft to its present elevation, was begun in 1936 and completed in 1937. Grouting of joints continued several months into 1938. Where the new concrete rests on the horizontal surfaces of the old base section and of the original crest, keying was provided by cutting notches in the old concrete, 16 in. wide, 8 in. deep, and 4 ft long, and by roughening the surface between notches, removing all of the original surface to an average depth of about $\frac{3}{4}$ in. The arrangement of notches was such that about 45% of the total area was notched and 55% merely roughened.

The treads and risers of the steps on the old downstream face were roughened, and in addition the risers, in the north one third and the south one third (approximately) of the face, were notched vertically. The surfaces of the siphon spillways were roughened.

The contraction joints of the old dam continue downstream into the new work, and a joint was provided in continuation of the aforementioned crack. Where the distance between the old joints exceeds 60 ft, the new section has intermediate joints, except in the conduit block. Some irregularities in spacing near the ends result from topographic considerations. Most of the blocks are 42 to 50 ft between joints, the minimum length of block is 30 ft, and the maximum 97 ft. The average of the 17 blocks along the 900-ft crest length is 53 ft.

The transverse joints of the original dam are keyed, with panels 15 ft wide, beveled at the edges, and continuous from near rock foundation to near the upper surface of concrete. This same arrangement was used in the new section, except that at each 50 ft of elevation the key is suppressed and a sheet steel horizontal grout stop spans the joint. Grout stops also were placed near the upstream and downstream limits of the joints.

In considering the joint along the downstream face of the original dam, the simple pouring of new concrete against the face, as a part of the pouring of each lift, was rejected at the start because of the obvious danger of developing undesirable shearing stress and opening cracks between the new concrete supported on the steps and the new concrete downstream which would shrink appreciably downward as cooling progressed in the lifts previously placed, and as compression increased in the lower concrete due to the increasing weight of the superposed concrete. The ideal would be to keep the new section out of contact with the old face until it was fully cooled, but this would be impossible because of the necessity of supporting the new concrete overhanging the old face. The design finally evolved provided a series of slots formed between the old and

the new concrete, averaging about 5 ft in thickness normal to the face and about 20 ft in width measured along the face, extending continuously from the foundation of the addition to the old crest level. Adjacent slots were separated by walls or ribs 2 ft thick, poured integrally with the adjacent mass concrete. The downstream face of the slot was so formed that the concrete filling the slot would key the old and new sections to transmit stresses in radial planes. In horizontal section, the slots in the middle portion of the dam were rectangular; those toward the ends were tapered, with the broad ends toward the abutments; this latter arrangement, combined with the aforementioned notches in the risers of the steps of the old face, provided keying for transmission of horizontal stress.

The ribs between slots were intended to carry very high unit stresses during construction, and were expected to crack, as most of them did. Cracks, which in general were mere hairlines or little more, formed across the ribs, normal to the slope of the face, from the outer corners of most of the steps, but there was no indication that these cracks extended into the adjacent mass downstream from the ribs.

A further tie between the old and the new work was provided by placing $1\frac{1}{4}$-in. square deformed reinforcing steel bars across the slotted joint, normal to the general direction of the slope of the face. There is one horizontal row of bars for each 5-ft lift, and the average horizontal spacing is about 2.5 ft. The bars are grouted into holes drilled 5 ft deep in the old concrete, and extend through the slots and 5 ft into the new concrete downstream from the slots.

To drain the longitudinal joint, horizontal drains extend along the original face with a vertical spacing of 5 ft, connecting to gutters in inspection galleries.

All of the new concrete was cooled by circulating water through embedded pipes of 1-in. nominal size on 5.5-ft spacing, in the same manner as in Boulder Dam. The slots were filled with concrete after the adjacent mass concrete was cooled, and then the concrete in the slots was cooled.

Finally, all joints (except, of course, horizontal ones) were grouted. This operation included the radial joints in the original dam, as well as the slot and the radial joints in the new section. Most of the grouting was done after practically all concrete was completed.

Apparently the objects in view in the design have been achieved; data so far obtained indicate that the dam as enlarged is behaving as it would if it had originally been built to its present size.

Basically, in the case of O'Shaughnessy Dam, the design of the longitudinal joint against the original downstream face, with the slots, ribs, and tie bars, resembles that used in the first enlargement of Assuan Dam in Egypt. In that work steel tie rods $1\frac{1}{4}$ in. in diameter, spaced 1 m apart horizontally and vertically, were set about 4 ft deep in the old cemented rubble masonry to project an equal distance into the new, and a 6-in. slot was left along the original face. The overhanging new masonry was supported on the tie rods and on 6-in. ribs spaced 49 ft apart. The slot was filled by grouting through perforated pipes after allowing time for equalization of temperatures in old and new masonry. The ribs appear to have been intended primarily to control the flow of grout, their functioning as supports being incidental.

Barton M. Jones,[77] M. Am. Soc. C. E. (by letter).[77a]—The excellent paper by Messrs. Paul and Jacobs presents the important phases of foundation preparation in an interesting and enlightening manner, well covering the ground, and along general lines of preparation by repair, rather than by wholesale removal, of rock. Such removal becomes time consuming and very costly when the cost of the concrete to replace it is added to the cost of the rock excavation. The parts of the paper covering recent developments in equipment and methods are of special interest.

The paper calls attention to the necessity of controlling the pressure in grouting operations and cautions against pressures that are too high. However, pressure ranges of 50 to 200 lb per sq in. for low-pressure grouting and 250 to 600 for high-pressure grouting are mentioned. It is clearly stated that there is danger of rock dislodgment with these pressures and that the utmost caution must be exercised in their application. This danger should be fully comprehended by those engaged in grouting operations as otherwise serious damage may be done to the foundation. Of course, the direction and position of the seams, and character of the rock, dictate the safe allowable pressures. Little displacement or damage could be done to a foundation in which the seams are nearly vertical, whereas, on the other hand, horizontal seams must be grouted with great precaution to avoid lifting portions of the foundation, and thereby progressively extending the area of the seams, and possibly disturbing the adjacent grouting that has already been done in other seams. One of the principal objections to the occurrence of a slight uplift is that it may extend the area upon which the grouting pressure acts with a resulting accumulative uplifting effect.

The writer desires principally to offer a general plea and some arguments in behalf of lower grouting pressures. It is safe to say that in many cases satisfactory grouting from the standpoints of bearing, uplift, and seepage can be accomplished at low and moderate pressures. The use of higher pressures may obscure the possibilities to be found in lower pressures. It is observable during grouting operations that most of the grout (possibly 99%) enters the grout holes and seams before the back pressure starts on its final upward jump. In other words, the high pressures and attendant hazards, whatever they may be, are related to a final spoonful of grout. The question may be asked, "Why use high and probably damaging pressures where low pressures can serve the purpose; and, once being convinced that the horizontal seams must, and can, be grouted with moderate pressures, just why should extremely high pressures be used on steeply inclined seams simply because the formation will permit?" There is reason for using high pressures in grouting deep and fairly tight seams in igneous rock formations where drilling becomes a larger part of the cost.

Uplift of horizontally stratified rock begins at surprisingly low pressures. For example, at Norris Dam it was the practice to grout the different seams in each hole separately, and the pressure in pounds per square inch was not allowed to exceed the depth in feet of rock over that seam. The advisability of this rule was verified by many observations of uplift. Grouting to a depth

[77] Cons. Design Engr., TVA, Knoxville, Tenn.

[77a] Received by the Secretary September 9, 1940.

of 40 ft was done over the entire base of the dam and power house. Over this area of shallow grouting the pressures were kept under 35 lb per sq in., and the highest pressures used on the project, under the completed dam, were from 125 to 150 lb. The numerous seams in the foundation under the dam ranged from close contacts to open seams or pockets in which the rock was separated by a foot or more. The cost of drilling and grouting (approximately $759,000) indicates that the work was both extensive and difficult. During an experience record of three years it has been found that the treated foundation is particularly sound and tight against seepage under the 200 ft of head against the dam, and there has been no evidence to indicate that the relatively low grouting pressures used were in any way inadequate or that higher pressures would have been better.

Uplift generally exists to some degree during grouting, and the need is to keep it under observation. The frequent occurrence of uplift was brought to attention by a special type of uplift gage originated, perfected, and used extensively at Norris Dam. Owing to its simplicity and usefulness, a description of the gage might be of interest. In principle, it detects uplift at the surface with reference to the underlying undisturbed rock. In its elementary form the gage is constructed by drilling a hole within the area to be grouted and several feet deeper than the surrounding grout holes. A substantial steel rod is lowered into the hole and securely anchored into the underlying rock at its lower end with cement mortar, or otherwise. A substantial steel yoke, the under side of which just touches the top end of the rod, is securely anchored to the surface rock by cementing the ends of both legs into holes drilled about a foot deep. Any upward movement of the rock surface carries the yoke upward and increases the gap between it and the top end of the rod. Certain refinements and precautions become evident in its use. For instance, grout might enter the hole from the seams being grouted, and to keep the rod free from the side walls it should be surrounded with a soft wrapping or some kind of flexible tube. The equipment must be sturdy to resist the hazards of a construction area such as truck wheels. The yokes may be made of 2-in. steel bar, and smooth brazed points will furnish rustless gaps between the yoke and top end of rod that can be measured with feeler gages to within 0.001 in., thereby providing accuracy with the necessary ruggedness and permanency while avoiding the many disadvantages of delicate indicator or other gages. Temperature changes in the rod are of no consequence in grouting operations, but for some other uses of the gage, such as measuring settlement of turbine foundations or foundation deflections under a dam, an allowance may be necessary to correct for the effect of the seasonal temperature variation on length of the rod.

The gages form a most useful aid in eliminating conjecture in controlling grouting operations. In using them, several should be located in the area to be grouted, and it is an easy matter to check and record the uplift each hour or oftener by simply measuring the gaps between the ends of the rods and the yokes.

The knowledge obtained from uplift gages cannot be put to full use unless the grouting is controlled and the pressures are carefully limited to the values

found safe for each area or group of holes. It is not safe to depend upon hand control of the pressure when reciprocating pumps are used for grouting. With this very satisfactory type of equipment it becomes necessary to have a dependable adjustable pressure regulator on the power end of the pump, and an adjustable pressure-relief valve on the grouting pipe. Sometimes it is also necessary to guard against excessive pressures when washing out seams preparatory to grouting.

In conclusion, it may be stated that on extensive or important grouting operations there is justification for continually checking the pressures that may be used with safety, and then to keep within those pressures by means of automatically regulated equipment. Rather than to exceed the safe grouting pressures in an attempt to force in more grout, a better procedure would be to use more holes. The ultimate cost may be less as much damage and further grouting may be avoided. It is believed that this discussion is a clarification and amplification of principles already set forth on grouting in the most excellent paper by Messrs. Paul and Jacobs.

P. E. Gisiger,[78] M. Am. Soc. C. E. (by letter).[78a]—During the past twenty years greater progress has been made in the art of designing and building large dams than during centuries before. The paper on the fundamentals of dam design by Messrs. Houk and Keener, who belong to a group of engineers responsible for a very large part of this progress, is, therefore, unusually authoritative. It is particularly valuable for those engineers who are not intimately and actively connected with the building of high dams because it permits them to visualize what has been accomplished and to what extent it is now possible to substitute exact reasoning in problems, which, a relatively short time ago, were left to guesswork or to the care of the so-called safety factors.

In 1935 an article was published in French[79] by Henri Juillard, which affords an interesting comparison of the most advanced thought on the subject in Europe and in the United States. It is believed that this article is valuable enough for a short digest of it to be presented as part of the discussion of this paper as well as of the Symposium in general.

Mr. Juillard confines himself to dams of the straight gravity and curved gravity type. He first questions the idea still held in some quarters that the massive straight gravity dam is the safest type of structure. The two points sometimes argued in favor of the straight gravity dams—(*a*) the possibility of a simple analysis of all forces and stresses, and (*b*) the absolute stability on account of the ever-present mass—are shown to be mistaken. The first contention is demonstrated to be faulty mainly on account of the twist action resulting from sloping abutments, but also because it could be rigidly correct only for the conditions of homogeneous material, uniform temperature, and foundations with elastic properties equal to those of the dam, all of which are never fully realized. The second contention is accepted for dams of moderate height, where stresses are not a factor of much importance. A high dam, how-

[78] Structural Engr., Pennsylvania Water & Power Co. and Safe Harbor Water Power Corporation, Baltimore, Md.

[78a] Received by the Secretary May 20, 1940.

[79] "Progress in the Construction of Large Masonry Dams," by Henri Juillard, *Le Génie Civil*, August 24, 1935, p. 178.

ever, might fail from shear or compression at its toe even if its weight were ample for external stability. In the final analysis, therefore, the safety of a high dam does not depend upon its mass, but upon the stresses that are developed, and can be resisted by the material of the dam itself and the underlying rock. From this it follows that no type of dam can be declared the "best" or "safest," but that for a given location one type may be better adapted, and therefore safer, than another type.

Mr. Juillard then states that originally a straight gravity dam with a width-height ratio of 85% was proposed for a high dam in Switzerland (the Spitallamm dam of the Grimsel development) but that finally a curved gravity or massive arch dam with a width-height ratio of 60% was adopted. It is emphasized that the main reason for this change was not the saving of about 130,000 cu yd of concrete, but the conviction that stress conditions would be more favorable for the curved dam.

Discussing concrete behavior during the setting and cooling period, Mr. Juillard emphasizes the importance of plastic flow of fresh concrete because without this property the various superimposed causes of volume change, such as heating, cooling, drying, and shrinkage, would result in an excessive number of cracks. Cracks are not quite unavoidable, but by proper subdivision into blocks and sequence of placing, their number can be substantially reduced, which, of course, is now recognized everywhere.

With regard to concrete mixes the importance of the proper proportion of fines in the aggregate is stressed. It is claimed that either a deficiency or an excess of fines will produce porous concrete which does not resist freezing, and that with the proper quantity of fines the least quantity of water will be required to produce workable concrete.

Mr. Juillard declares himself in favor of especially rich facings for the upstream as well as the downstream face of a dam. This is apparently not in agreement with most recent American practice and may be open to question with regard to the importance of having homogeneous material throughout the dam, apart from the economical side of this point.

The most impressive part of Mr. Juillard's paper is the final third wherein the need for continuous observation and inspection of dams after completion is discussed. It would seem natural that the investment represented by a large dam should call for close and constant check of its condition and behavior. It is rather astonishing, therefore, how long it has taken before the need for such supervision was recognized, and before the attitude was overcome that a masonry dam, after the last batch of concrete has set, is an absolutely stable, immovable, and noncorrodible object which does not require any further care.

The feature to which attention was given first, and for a long time almost exclusively, was leakage, especially if it amounted to more than had been expected. In the more recent dams, which have inspection galleries, intercepting tunnels, riser pipes, and other devices, it is easier to get an idea of the probable total leakage than in older dams where such devices are usually absent and the total loss of water must be judged from what comes to daylight at the downstream side.

In some dams devices have been installed which record the quantity of leakage water passing a given point. Mr. Juillard mentions apparatus which, in addition, records the percentage of lime carried in solution by this water. The latter point is emphasized to illustrate the fact that concrete is not an inert material, since a considerable tonnage of lime can be removed annually from a large dam.

Apart from cases of extreme leakage or very poor concrete, however, judgment on the continued structural soundness of a high dam must be based mainly on observation of its deformation under load and the influence of temperature. This may have been recognized quite early, but the practical difficulties of measuring deflections on a dam are much greater than, for instance, measuring deflections of a bridge. Therefore, such measurements are a fairly recent development.

The methods of measuring deformations used in Europe were described by Mr. Juillard in a paper[80] read before the World Power Conference in Stockholm, Sweden, in 1933. In the paper cited herein[79] he stresses the necessity of continuous, regular observation for the purpose of gaining complete knowledge of the factors affecting a dam and the availability of apparatus permitting the making of such observations at relatively little expense. He cites as an example the previously mentioned Spitallamm dam of the Grimsel development where the horizontal movement of a number of points is observed twice weekly by means of pendulum indicators, permanently built in, which give an accuracy of $\frac{1}{20}$ mm.

It is also stressed that these measurements, as well as others on recent dams, indicate a complete elastic behavior under load, contradicting some earlier opinions based on incomplete observation to the effect that dams undergo a permanent deflection. It is important, of course, to separate the effects of temperature from those of load. Temperature effects predominate near the crest of a dam and are subject to daily change. In 1939, Douglas McHenry and Roy W. Carlson, Assoc. M. Am. Soc. C. E., presented[81] results of observations on the Norris Dam which illustrate this point and indicate the fact that Mr. Juillard's ideas are in agreement with recent practice in the United States.

Mr. Juillard goes on to state that, although the measurements on the dam proper are those of most importance, the rock under and next to the dam must also be considered. A large number of complex questions remain to be solved in this respect. It is not known, for instance, what part of the deformations caused by the weight of the dam as well as the stored water is permanent or elastic, how this may change with different kinds of rock, how changes in surface temperature due to the covering with water affect the rock, how deep these influences go, and how long it takes to establish a new equilibrium.

Every one of these uncertainties is an additional reason for continuous, detailed observation of an important dam after its completion. The proper safeguarding of investment should be reason enough for the relatively small outlay

[80] "Report No. 12" by Swiss National Committee, 1st International Cong. on Large Dams, Stockholm, 1933.

[81] "Measuring Dam Behavior," by Douglas McHenry and Roy W. Carlson, *Engineering News-Record*, March 30, 1939, p. 58.

necessary. A much weightier reason is the safeguarding of life and property. In this respect, it may be reasonably claimed that the few disasters which have occurred could have been greatly mitigated, if not altogether avoided, if regular, frequent observations of structural movement on those dams had been taken.

Finally, Mr. Juillard stresses that only continuous observation on the completed structures will indicate whether the design assumptions and methods were correct and will stand the test of time and that only by comparison of actual behavior with design theory can further improvements be expected in the efficiency of design and perfection of material.

The foregoing tends to show that ideas on the design and construction of large masonry dams have, in recent years, moved toward the same conclusions in Europe as well as the United States. Twenty-five or thirty years ago masonry dams of European design showed a tendency to conform to a set type, without much regard to site conditions. They were usually dimensioned to satisfy rigid safety requirements (such as the "Lévy condition") based on strictly two-dimensional stress analysis and contained often elaborate expansion joint and drainage provisions. In America, on the other hand, development went forward without much benefit or hindrance by precedent. Uplift assumptions, which largely govern the mass of a gravity structure, and provisions for drainage and treatment of joints were generally left to the judgment of the designing engineer and showed great variation.

With the increase in number and size of dams that were designed and built since 1920, research into the design problems was greatly stimulated and brought about a vastly increased knowledge about the way in which the external forces acting on a dam affect its internal stresses and deformations. This general clarification of ideas produced more uniformity of design. Mr. Juillard's paper, summarizing the same period of development in Europe, shows that at the present time the basic considerations as well as methods of design on both sides of the Atlantic are practically the same. The importance of twist action caused by sloping abutments, the combination of cantilever and arch action in curved gravity or massive arch dams, and the influence of elastic deformation of the underlying rock are given the same relative degree of emphasis by Mr. Juillard as by the most authoritative American thought which is embodied in the papers presented for this symposium. Beyond that the writer believes that Mr. Juillard's insistence on periodical and systematical check of the behavior and performance of a dam after its completion is an important and valuable contribution to the subject matter presented on this occasion.

JOSEPH A. KITTS,[82] ESQ. (by letter).[82a]—As an index of the present state of progress in concrete manufacture and control as applied to dams, the paper by Mr. Tyler comprehends only a very recent school of thought on the subject, as indicated by its appended list of references. Although it admits that there are some scientific questions yet to be solved by this school, it says (see "Synopsis") that "knowledge concerning properties and behavior of concrete * * * has been enormously increased during the last few years,"

[82] Cons. Engr. (Joseph A. Kitts Co.), San Francisco, Calif.

[82a] Received by the Secretary June 6, 1940.

being in complete accord, on this last point, with each succeeding report of the kind in the past.

On the other hand, the concrete manual mentioned in the Appendix (2)[82b] shows that this school of thought has very little appreciation and knowledge of the fundamental physics and basic principles of concrete mixtures brought forth in twenty-two centuries of laborious and costly research since about 250 B.C., when Archimedes discovered how to determine the absolute volume (the basic measure) of an irregular solid by liquid displacement. Instead, this school has adopted a false physics using weight as the basic measure. Its false physics has led to erroneous test results. Its erroneous test results indicate a new and simplifying principle of mixtures, although definitely and widely shown to the contrary 40 years before.[83] It adopts this pseudo-principle as its "fundamental rule." Its false physics and false principles are written into its specifications and naturally affect the practical features of control. The result has been an enormous waste of cement and aggregate, and the quoted "enormous increase of knowledge concerning the properties and behavior of concrete" is based on, and colored by, this false physics. Mr. Tyler does not offer anything to correct the situation indicated in the foregoing, nor does he appear to appreciate it.

In a series of seventeen articles,[84] the writer has coordinated all the known and authoritative fundamentals of concrete mixtures in complete detail of mathematical analysis and expression, giving sources, authorities, and practical application. The absolute volume measure of Archimedes was used as the only logical basis on which to coordinate: (*a*) The matrix-to-voids principle of François Coignet;[85] (*b*) the absolute volume composition law of René Feret;[83] (*c*) the grading principle of Sanford E. Thompson and the late William B. Fuller,[86] Members, Am. Soc. C. E.; (*d*) the water-cement strength law and fineness modulus principle of Duff Abrams,[87] M. Am. Soc. C. E.; (*e*) the cement-content strength law and fineness modulus law of the writer;[84] and (*f*) various miscellaneous principles by others. These are the complete fundamentals of concrete technology; there are no successful or basic alternatives. With this history of concrete research and the coordination of basic principles of concrete mixtures before them, the sponsors of Mr. Tyler's school of concrete control strangely went out of the way to adopt false physics and the disproven theory that water content and slump remain constant as the cement content is increased.[83]

Absolute Volume the Basic Measure.—"The absolute volume is the basic measure of concrete ingredient proportions and characteristics." That is the fundamental law of concrete physics and technology.[84] Cement, aggregate, and concrete tests and concrete research, disregarding this law, have been

[82b] Numerals in parentheses, thus: (2), refer to corresponding items in the Appendix of the paper.

[83] *Bulletin de la Societé d'Encouragement pour l'Industrie Nationale*, by René Feret, Vol. II, 1897, p. 1593.

[84] "Coordination of Basic Principles of Concrete Mixtures," by Joseph A. Kitts, *Concrete*, November and December, 1931; April, 1932, to April, 1933; and February to April, 1934.

[85] "Coignet Béton," by the late Q. A. Gillmore, M. Am. Soc. C. E., Washington, D. C., Government Printing Office, 1871.

[86] *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 67.

[87] "Design of Concrete Mixtures," by Duff A. Abrams, *Bulletin No. 1*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., 1919.

almost worthless, and this disregard accounts for the fact that man's 10,000 years of concrete making had not evolved a complete concrete technology until the discovery of the cement-content strength laws, in 1929. Until this fundamental law of the physics of concrete materials and mixtures is used, there can be no concrete technology worthy of the name; and, until then, research of the properties and behavior of concretes will continue to be largely barren waste and the source of costly errors.

Weight is the most accurate and flexible means of measuring concrete ingredients, and should be required for the measurement of the solid ingredients of concrete for major structures, and for structural concrete in general. It is an amazing fact of concrete history, however, that, where the volumetric batcher, basket, barrow, or bin is used as the means of measure, the volume of the measure (not the absolute volume of material in the measure) has been the basis of proportions, and is so today, on 75% of all concrete specified. As in the case of volume as the means of measure, when weight has been adopted as the means of measure, it has also been adopted as the basis of measure.

Confusion of Measures.—Mr. Tyler treats almost all of the particulars of concrete control except the fundamental physics of materials and mixtures. The writer contends that there can be no sound understanding of any feature of concrete control until the fundamental physics of materials and mixtures is understood; and the fundamental physics cannot be understood until the basic measure is recognized. Accordingly, the confusion of measures appearing in sources cited in the Symposium should be considered. It should also be noted that centuries of civil engineering thought, or lack of it, are responsible for this confusion.

In this school of thought the water-cement ratio is expressed by weight instead of volume of water to absolute volume of cement. Thus, a water-cement ratio of 1.0 by weight means a water-cement ratio of 2.8 and 3.2 by volume, for cements having specific gravities of 2.8 and 3.2, respectively. Obviously, the weight basis is indefinite. On the other hand, volume of water to absolute volume of cement is a definite ratio and conception, regardless of the specific gravity of the cement, and engineers know that 1 cu ft of water added to 1 cu ft absolute volume of cement, for example, contributes the sum, or 2 cu ft, to the concrete mix.

Fineness of cement is expressed as specific surface, or square centimeters, per gram of cement, instead of square centimeters per cubic centimeter absolute volume of cement.

Fineness modulus of aggregate is determined from the proportionate weights retained on the sieves, disregarding natural differences in the specific gravities of the fine and coarse parts of the aggregate.

Unit weight of aggregate is determined as the weight of dry-and-rodded material filling the standard container of unit volume. The volume of the container is the basis of measure, not the absolute volume of material contained.

Absorption and moisture contents of aggregate are determined as weight of water to weight of aggregate (0.02 absorbed water by weight, in the case of Haydite aggregate having an apparent specific gravity of 1.0, is 0.02 cu cm of

water per cubic centimeter absolute volume of Haydite, and is 0.053 cu cm of water per cubic centimeter absolute volume of rock having a specific gravity of 2.65).

Specific gravity of solids is determined as weight of solid to weight of water displaced by the solid, or, weight of solid in grams to absolute volume of solid in cubic centimeters. The basic measure of the material is the absolute volume. This is the simple physics developed by Archimedes some twenty-two centuries ago.

One sees, then, as general and standard practice, the indiscriminate use of loose-moist, loose-dry, and dry-rodded bulk volume, and the volume of the container, weight, and the absolute volume as entangled bases of proportions and characteristics. This is not physics, science, engineering, or logic.

Using weight, the gravitational characteristic, as the basic measure of proportions to produce an absolute volume of concrete, when it is known that the absolute volume of concrete is the sum of the absolute volumes of the ingredients, seems too obviously unscientific for an engineer to accept. On the other hand, since absolute volume has not been accepted as the basic measure of proportions in twenty-two centuries, perhaps it is too abstruse an idea for the layman in concrete physics and technology to grasp. Another point of view is that concrete manufacture is a simple and practical art, and not a complex science. The writer observes, however, that the science works —practically, efficiently, and economically; the simple and practical art does not. Mr. Tyler's point of view, as stated in his "Conclusion," is that "The art of building concrete dams is becoming a science." Lack of appreciation of the absolute volume as the basic measure of concrete proportions is still holding it in leash as it has concrete technology for twenty-two centuries.

False Theories Due to False Physics.—Under the heading, "Concrete Control: Proportioning," Mr. Tyler states that "water content of concrete at fixed workability changes little with cement content." In other words, with a given mix, as aggregate is replaced with an equal absolute volume of cement, the slump, consistency, or workability of the mix remains the same, or changes little, according to Mr. Tyler.

In 1897 the noted French engineer and dean of concrete physicists, René Feret, published the results of an elaborate series of tests, made by him, of fine, medium, and coarse sand-cement mortars of normal consistency, in which he varied the absolute-volume cement content in ten stages from about 3% to about 53% by volume of the resulting mortar.[83] A table of the results can be found in a publication by Frederick W. Taylor and Sanford E. Thompson.[88] The water per unit volume of concrete for Feret's fine sand "*D*" follows the semilogarithmic equation

$$\log W = 0.33\, C - 0.58 \text{ (approximately)} \qquad (7a)$$

in which W is the volume of water per unit volume of concrete, and C is the absolute volume of cement per unit volume of concrete. His coarse sand "*G*"

[88] "Concrete, Plain and Reinforced," by Frederick W. Taylor and Sanford E. Thompson, 2d Ed. (1909), pp. 136–137.

follows the straight-line equation

$$W = 0.58\,C + 0.1 \text{ (approximately)} \qquad (7b)$$

Feret's medium sand lies between the two foregoing, and their three graphs approach neat cement mortar composed of: 0.534 cement, 0.414 water, and 0.052 air. Thus, at 5% cement, the water content ranges from 13% for coarse sand to 27% for fine sand; and, at 53% cement, or neat mortar, the water is 41%; that is, the water content for neat cement mortar of a given consistency is shown as 52% to 215% greater than that for a sand-cement mortar containing 5% absolute volume of cement (2.75 sacks per cubic yard).

The writer has made concrete and neat cement mortar tests of Bonneville type cement which show the relations of cement and water contents of mixtures of a given slump, as indicated in Table 14.

TABLE 14.—WATER CONTENTS FOR NEAT CEMENT MORTARS COMPARED WITH THOSE FOR LEAN CONCRETES

Standard slump, in inches	Concrete, 1¼-In. Maximum		Neat Cement Mortar	
	Cement	Water	Water	Cement
	Proportionate Absolute Volume			
1	0.073	0.118	0.422	0.549
2	0.073	0.124	0.432	0.542
3	0.073	0.132	0.442	0.536
4	0.073	0.140	0.452	0.529
5	0.073	0.151	0.462	0.522
6	0.073	0.163	0.472	0.515
7	0.073	0.181	0.484	0.507
8	0.073	0.216	0.499	0.496

In changing from 1 bbl of cement per cubic yard of concrete to neat cement mortar, it is seen by Table 14 that the water content for a given slump is increased 257% for the 1-in. slump and 131% for the 8-in. slump. Any other calcareous cement will show values of a similar order.

Table 14, for 1¼-in. maximum size of aggregate, confirms Feret's tests of mortars made 40 years earlier. Mr. Tyler's qualification that the water changes little with cement content is quite true if the cement content is changed only a little; concrete manufacture, however, comprehends cement contents from 2% to 55% of the concrete volume. The false theory is due to test errors characteristic of the false physics of using weight as the basic measure of ingredient proportions and characteristics.

Laws of Proportioning Are Needed for Any Maximum Size and Grading of Aggregate, Strength, and Slump of Concretes.—In 1931, the writer analyzed the water and cement content relations shown by data of the Portland Cement Association.[89]

For any given maximum size of aggregate and slump of concrete, the change of water content with change of cement content was discovered to follow the

[89] "Design and Control of Concrete Mixtures," Portland Cement Association, 2d Ed., January, 1927.

semilogarithmic equation,

$$\log W = m\,C + k \quad \text{(8)}$$

(in which m and k are constants depending on the maximum size and fineness modulus of aggregate and the slump of concrete), provided the fineness modulus of grading follows the equation:

$$f = a \log C + b \log D + c \quad \text{(9)}$$

in which D is the maximum size of aggregate and a, b, and c are constants.

Although there are occasions in which Eq. 8 is convenient to use, the constants m and k for a given maximum size or slump do not vary in accordance with any simple mathematical expression as the maximum size or slump is varied.

In 1929, the writer discovered that the law of the cement content for any strength, for a given maximum size and slump, is expressed by the equation

$$\log C = d\,S + e \quad \text{(10)}$$

in which S is the compressive strength at a given age and d and e are constants depending on the maximum size and slump. It was further found that e, in Eq. 10, varied as the slump, or

$$e = g \text{ slump} + h \quad \text{(11}a\text{)}$$

and as the log of the maximum size, or

$$e = n \log D + j \quad \text{(11}b\text{)}$$

in which g, h, n, and j are constants.

This family of cement-content laws is supplementary and complementary to the Abrams water-cement ratio law; it is closely accurate for strengths greater than 1,000 lb per sq in. at 28 days and indicates excess cement at desired strengths less than 1,000 lb per sq in. It is too well founded and too adequate to have been abandoned for a new theory which the competent tests of the dean of concrete physicists had shown to the contrary, 40 years before, and which the comprehensive data herein cited[89] had shown to the contrary more than 10 years before.

Theoretical Grading and Proportioning.—Mr. Tyler states that "Too close adherence to theoretical gradings usually leads to undersanded and harsh concrete mixtures," and "It is seldom if ever possible to calculate an aggregate grading and proportion a concrete mix without experimental data or actual trial batches of concrete as a guide." There are great differences in the shape of particles of crushed rock from various sources, of course. Angular, elongated, and lens-shaped particles demand a lower fineness modulus of grading (and more cement) than do rounded gravels. However, a grading suitable for natural gravel from one source will almost invariably be found suitable for gravel from any other source, provided the grading is on the basis of absolute volume measure. If theoretical gradings produce undersanded, oversanded, and harsh mixtures, and mixtures which segregate badly, the criteria, theory, or technique of grading is obviously wrong.

There are considerable data, of normal standard portland cement and crushed rock and gravel concretes, which supply good criteria of the principles of grading and proportioning as a basis of experimental work on any new project. Knowing the physical characteristics of the materials and the principles involved, gradings and mixtures can be calculated on the basis of the criteria with a high degree of certainty as to resulting workability, slump, yield, and strength. Not knowing the physical characteristics and the principles involved, the only recourse is trial mixtures and experiment, which, in turn, depend upon principles and technique. Of course, the physical characteristics of the materials are not fully established until they are tested in the concrete mixtures, and no dam concrete should be produced without preliminary tests and subsequent laboratory control on the job. Laboratory control means continuous testing of material characteristics, and occasional and sometimes frequent changes of the measured proportions, to maintain the basic structure of the concrete composition in absolute volumes of cement, water, and stone particles of various diameters, as physical characteristics of the materials vary. Sometimes, also, changed conditions require modifications of the basic structure of the concrete composition; but to say that "It is seldom if ever possible to calculate an aggregate grading and proportion a concrete mix without * * * trial batches * * * as a guide" points to one or a combination of the following causes of failure: (1) Wrong criteria, (2) false physics, (3) false theories, (4) wrong characteristics, (5) graphical approximations, (6) erroneous equations, (7) erroneous technique, and (8) the major source of failure in grading, proportioning, and control is lack of appreciation of absolute volume as the basic measure of ingredient proportions and physical characteristics.

Many Types of Grading Are Practicable.—The average pit run, although generally somewhat oversanded, may be the most practicable and best grading from the view of engineering economy. If merely 5% of the material is finer than the No. 100 sieve and the remainder is sound, there is generally little reason for wasting any of it. If there is 55% finer than the No. 100 sieve, there would then be justification in wasting 50% of the pit run, since such excess of fine sand is merely an adulterant to the water-cement mortar, reduces the strength and density of the concrete, and generally costs less to waste than to use. There have been many engineering crimes committed in the name of grading, at added expense and sometimes with loss of quality of the concrete. "Soaking" the contractor to produce 5,000-lb concrete by wasting good aggregate, when the specification merely calls for 2,500-lb strength, is a practice which automatically increases the contract price with little benefit to the owner. From the point of view of national economy, it would appear to be better engineering economy to allow the contractor to have an extra million dollars of profit in his pocket rather than have it poured out and forever lost in an enormous and unwarranted waste pile.

The ideal grading of the coarse aggregate from $\frac{D}{10}$ to D is no doubt a straight line as proposed by Messrs. Fuller and Thompson, in 1907. An ellipse from

the No. 100 sieve, or 0.0059 in. to $\frac{D}{10}$ and tangent at $\frac{D}{10}$ to the straight line from $\frac{D}{10}$ to D, is perhaps the ideal sand to go with the ideal coarse aggregate. The equation of the ellipse is

$$y^2 = \left(\frac{b^2}{a^2}\right)[a^2 - (a - x - 0.0059)^2] \quad \text{(12)}$$

in which

$$a = \frac{y' (0.1\,D - 0.0059) - m\,(0.1\,D - 0.0059)^2}{y' - 2\,m\,(0.1\,D - 0.0059)} \quad \text{(13a)}$$

$$\frac{b^2}{a^2} = \frac{(y')^2}{[2\,a\,(0.1\,D - 0.0059) - (0.1\,D - 0.0059)^2]} \quad \text{(13b)}$$

and x is the sieve aperture, y is the percentage passing the given sieve, y' is the percentage passing at $x = \frac{D}{10}$, m is the slope of the straight line expressed as a percentage, D is the maximum sieve size of the aggregate, and a and b are constants for the given maximum size and point of tangency. This is very simple to use when one becomes acquainted with it, and gradings of any given fineness modulus can be determined without much trouble.

The Bolomey Curve[90] is simply a modified parabola of the proportion passing and can be determined for any fineness modulus by varying the value of N in the equation for the proportion retained

$$r = N\left(1 - \sqrt{\frac{d}{D}}\right) \quad \text{(14)}$$

in which r is the proportion retained on the given sieve of d opening, and D is the maximum sieve size of the aggregate.

The Talbot Curve described by Mr. Tyler (Eq. 4) should be written in the form:

$$r = 1 - \left(\frac{d}{D}\right)^n \quad \text{(15)}$$

in which n is an exponent, and not a "factor" as stated.

The Fineness Modulus Is the Practical Tool of Grading.—The graphical method of determining grading proportions is obsolete, slow, tedious, and inaccurate. By use of the fineness modulus and tabular values of the particular grading curve used, the proportions of 3 and 6 aggregate parts can be calculated in 3 and 6 min, respectively. Slaves to inflexible graphs are obviously rule-of-thumb workers in concrete technology. The graph as an illustration of the equation is useful, of course, but its use as the only means of determining values gives it a fictitious importance which generally overlooks the principles of variation involved. Eqs. 4, 12, 13, 14, and 15 each must comprehend a large number of curves whose fineness moduli need to correspond to Eq. 9, depending upon shape and maximum size of aggregate, and upon the cement

[90] "Contrôle des Qualités des Ciments," by J. Bolomey, *Travaux*, Vol. 21, No. 54, June, 1937, pp. 256–265.

content. For rounded gravel aggregate, the fineness modulus of the grading curve (see Eq. 9) should follow the equation

$$f = 1.1 \log C + 2.75 \log D + 2.31 \quad \text{(16)}$$

in which C is pounds of portland cement per cubic yard and D is the maximum square-hole size of the aggregate. If crushed stone or slag is used as coarse aggregate, the value of f should be reduced 0.25, and 0.40 if the crushed material consists of unusually flat and elongated particles. If stone screenings are used as fine aggregate, the value of f should be reduced 0.25. Other conditions may require a reduction of 0.50 and, again, may permit an increase of 0.25 or more.

The fineness modulus of aggregate, as a means of grading and proportioning, was presented by Professor Abrams[87] in 1918, and offered a more accurate and more expedient method than the tedious graphical one. Its sponsors in 1918, however, rejected it in 1928, instead of perfecting the technique of its use. As a consequence, the value and utility of this long-needed and essential physical characteristic of an aggregate has not been generally appreciated. Too little attention has been paid to the fact that the fineness moduli of the parts of an aggregate between the standard sieves in Col. 1 following are as shown in Col. 2:

Column 1	Column 2	Column 1	Column 2
No. 100–No. 50	1	$\frac{3}{16}$ in.– $\frac{3}{8}$ in.	6
No. 50–No. 30	2	$\frac{3}{8}$ in.– $\frac{3}{4}$ in.	7
No. 30–No. 16	3	$\frac{3}{4}$ in.– $1\frac{1}{2}$ in.	8
No. 16–No. 8	4	$1\frac{1}{2}$ in.– 3 in.	9
No. 8–No. 4	5	3 in.– 6 in.	10
		6 in.–12 in.	11

Accordingly, the fineness moduli of the parts of a theoretical or actual grading curve from 0–$\frac{3}{16}$, 0–$\frac{3}{4}$, 0–$1\frac{1}{2}$, or $\frac{3}{16}$–$1\frac{1}{2}$, etc., can be calculated readily. If it is desired, for example, to combine normal $\frac{3}{4}$-in.–$1\frac{1}{2}$-in. and $1\frac{1}{2}$-in.–3-in. job aggregates for a straight-line grading, it is evident that, theoretically, the $\frac{3}{4}$ in.–$1\frac{1}{2}$ in. should be 33%, and the $1\frac{1}{2}$ in.–3 in., 67%; and the theoretical fineness modulus of the $\frac{3}{4}$ in.–3 in. is, therefore, $(0.33 \times 8) + (0.67 \times 9)$, or 8.67. If the actual fineness moduli of the job aggregates are 7.95 and 9.07, the job proportions by absolute volume are determined by simple proportion (as shown by Professor Abrams[87]): $\dfrac{K - T}{K - F} = \dfrac{9.07 - 8.67}{9.07 - 7.95} = 0.357$ of $\frac{3}{4}$-in.–$1\frac{1}{2}$-in. aggregate, and $1.0 - 0.357 = 0.643$ of $1\frac{1}{2}$-in.–3-in. aggregate.

With this key, its elaboration should be obvious to any average mathematician familiar with the most simple operations of algebra. It can be used with the grading of the average pit run or with the most elaborate grading equation. The fineness modulus is a characteristic of the whole and the parts of any grading equation and is complementary to it. It is a physical characteristic quite as important in its utility as the specific gravity. Concrete technology would not be complete without it. Concrete control demands its constant use for efficient control.

Ample Slump Is Important.—Mixtures can be too dry for the conditions of use. Dry mixtures of 3-in. slump, and less, trap air and require an excessive amount of puddling work, and the volume of honeycomb increases with the dryness. Dry mixtures are the most permeable, medium mixtures the least permeable, and wet mixtures are less permeable than dry mixtures. The tendency since 1918, on dams and elsewhere, has been to make concrete too dry. Less than 3-in. slump is not warranted by the theoretical but questionable improvement of quality, nor by engineering economy. Making concrete so dry that it will not flow in chutes on 40% grade is abandoning the practical and economical for a questionable theory which is more costly. Experimentation too often looks at the quality obtained under laboratory conditions without regard to the quality per unit of cost on the job. Laboratory results too often disregard the labor and time element. During all the history of concrete there have been three schools of consistency: The Drys, the Wets, and the Moderates. The "old army specification" was "the consistency of fresh cow dung," as recommended[91] by the late Gen. Q. A. Gillmore, M. Am. Soc. C. E., about 1863. The terrible examples of honeycombed and patched concrete, since the inception of the Abrams water-cement ratio-strength law, suggest that practical lessons from a cow should supplement a theoretical knowledge of the Abrams law.

Since 1924 the writer has made many observations of the slumps and other conditions of mixture which produced satisfactory workability. The "optimum slump" for hand or manual puddling, tamping, and hammering (and flow in metal chutes on 40% grade) was found to be as given in Table 15. Mechanical

TABLE 15.—OPTIMUM SLUMPS

Maximum size (inch) (square aperture)......	$\frac{3}{16}$	$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{1}{2}$	3	$4\frac{1}{2}$	6
Optimum slump (inch)................	8.6	8.4	8.0	7.3	6.0	5.0	4.1

vibration will permit 1 in. less slump in the case of narrow and difficult sections in structural concrete, 3 in. less in massive concrete, and 4 in. less in pavements; but a reduction to less than 3-in. slump is a source of increased and unavoidable defects and increased cost. In general, concrete should be not only plastic but fluid, for the optimum results. It should flow in a smooth, metal, parabola-shaped trough on a 40% slope (1 on $2\frac{1}{2}$).

Conclusion.—Much more should be said regarding the practical considerations covered by Mr. Tyler's paper, as well as the scientific and technical. Mr. Tyler's school of concrete control is producing most excellent concrete, without doubt; but, also without doubt, that school of thought is using false physics, false theories, and practical ideas colored by them, at great expense and waste. No one person is responsible: The profession, the civil engineering colleges, the Society, every individual engineer—each is equally responsible for the chaos which has existed and still prevails in the accepted knowledge of concrete physics and technology. Until the fundamental physics of material

[91] "On Limes, Hydraulic Cements and Mortars," by Q. A. Gillmore, *Professional Paper No. 9*, Corps of Engineers, U. S. Army.

characteristics and the laws of concrete mixture composition and quality are understood, there can be no prideful progress in concrete control on dams or elsewhere.

S. O. Harper,[92] M. Am. Soc. C. E. (by letter).[92a]—The summarization, by Mr. Steele, of the practice in the art of building concrete dams, is a valuable contribution to the literature on dams. The painstaking care evident in the assembly of Table 11 and the supporting comments in the Appendix make of that section a most valuable reference. Comments made herein are listed under the same headings as appear in the paper.

Joint Spacing.—The spacing of vertical contraction joints so that joints occur at sudden changes in the foundation profile in order to minimize the formation of cracks, as discussed by Mr. Steele, is theoretically sound but usually impractical for various reasons. In the first place, the foundation conditions cannot be predicted with sufficient accuracy, prior to actual uncovering, for predetermination of the spacing of joints such that they will occur at the foundation breaks. Secondly, the usual irregularities encountered are seldom in directions or on straight lines that lend themselves to the location of contraction joints at the points of irregularity. Furthermore, the additional cost of variable size in forms as compared with uniform dimensions grows to a sizable value when account is taken of the construction delays and extra operations imposed by nonuniformity in procedure. In many cases material improvement can be effected in irregular foundations by a little expenditure for additional excavation. The undesirable effects of uneven foundation conditions in contributing toward cracking can be greatly minimized by suitably placing concrete in the depressions to build up the foundation to a general level and by adequate temperature control of concrete so placed in advance of general construction.

Slots Versus Grouted Joints Versus Open Joints.—The conclusion of Mr. Steele that the slot method of joint construction might be more satisfactory than the grout-film method is further substantiated when considering the normal variation in temperature of the concrete in thin dams constructed in extremely cold climates.

Very meager data are available of actual measurements of such temperature variations. Eight embedded resistance thermometers located across two transverse sections of Seminoe Dam in Wyoming, 21 ft from the top of the dam where the thickness is 21 ft, varied from an average of 24° in midwinter to 68° in midsummer. The mean annual air temperature at this location is 41°.

The contraction joints of Seminoe Dam were grouted in the early spring, as soon as the measured temperatures in the interior of the dam were more than 32°. Although every effort was made to grout the joints at the most favorable time, the average temperature of the eight thermometers at the top of the dam was 45° by the time this region was grouted. A reasonable estimate of the minimum closure temperature where contraction joints are grouted is about 40°.

Estimates of the range in temperature of a thin arch dam being considered for construction in a region in Colorado where the mean annual air temperature

[92] Chf. Engr., U. S. Bureau of Reclamation, Denver, Colo.

[92a] Received by the Secretary August 26, 1940.

is 32° indicate that where the thickness of the dam is 20 ft the average concrete temperatures will vary from 5° to 53°. Severe stresses would be imposed upon this arch subjected to maximum water load during periods of minimum temperature if closure were made at 40°. It is estimated that closure could readily be made with slots with mean concrete temperatures as low as 25°, which, in this instance, would be a considerably more favorable closure temperature and would obviously be difficult to accomplish with grouted joints.

Bonding Joint Surfaces.—Mr. Steele's statement that "The adequate bonding of horizontal and inclined joint surfaces has been one of the most difficult problems to solve satisfactorily * * *" is a very pertinent observation. The importance of this factor in present-day construction is well illustrated by the fact that the cost usually charged against horizontal joint treatment, commonly termed joint cleanup, is still an appreciable percentage of the total cost of mass concrete in place. At Grand Coulee Dam, where the final sandblast method of cleanup is being used, the actual net cost of cleanup for 1,312,941

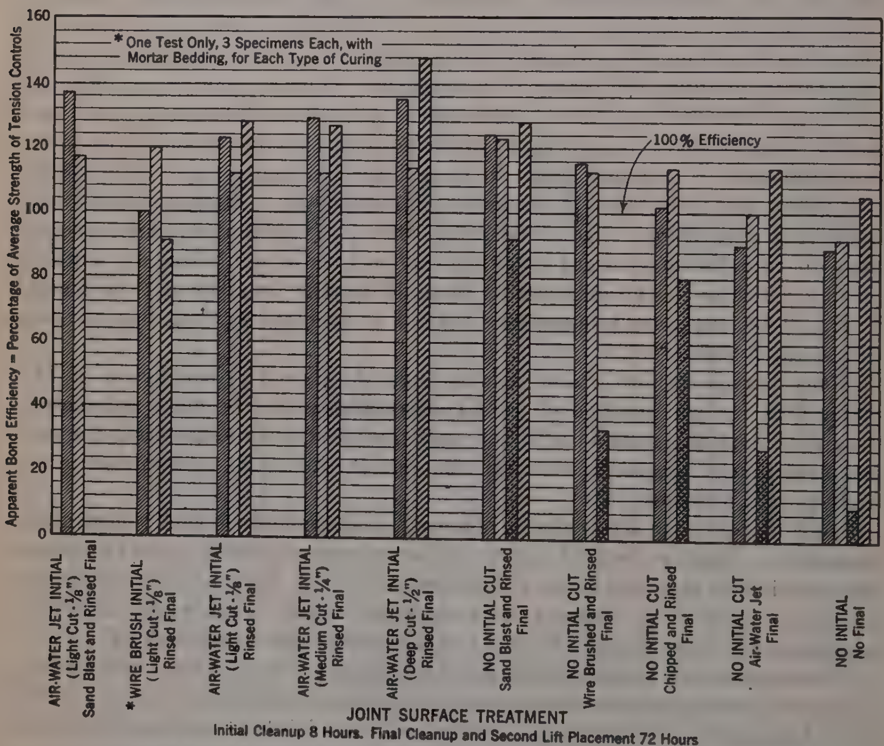


FIG. 26.—RELATIVE BOND EFFICIENCIES FOR VARIOUS COMBINATIONS OF CLEANUP AND CURING METHODS (AVERAGE RESULTS)

cu yd of concrete placed during the first half of 1939 was about 23 cents per cu yd. This cost was obtained through the courtesy of Edgar F. Kaiser, Assoc. M. Am. Soc. C. E., project manager for Consolidated Builders, Inc., contractors on the completion of Grand Coulee Dam, who were able to reduce the sandblast

cleanup cost to the reported 23 cents from a considerably higher value, only after extensive research to develop suitable equipment and operating technique. Less than one third of this cost fully covers the sandblasting operation and its maintenance. The remaining two thirds represents the cost of removing construction wastes, mucking, final wash-down and maintenance incidental to these operations which are common to all thorough cleanup procedures under present-day methods of construction. It is apparent that the development of joint cleanup methods which will insure satisfactory results more economically presents a promising field for investigation.

Mr. Steele has indicated certain precautions to be observed and methods to be followed in order to "solve satisfactorily" the joint cleanup problem, but it appears desirable to define what can be considered as satisfactory results. Certainly there is no object in striving for bond strength and watertightness at the horizontal construction joints in excess of the tensile strength and impermeability of the concrete mass. From a study of Fig. 26, which summarizes the results of very carefully conducted laboratory tests on construction joint bond tests, it appears that 100% bond strength, or more, can be obtained by any one of several methods of treatment. However, these results should be interpreted in terms of field conditions, which require a margin of safety and a human-element-proof set-up to provide for the uncertainties of construction variables, and not in terms of the ideal laboratory control under which the tests were made.

In appraising these test data, the following items apply:

1. The bond strength was measured in direct tension;
2. Efficiency was based on the average strength of solid tension control specimens;
3. The test specimens were 8-in. by 16-in. cylinders with mid-sections reduced to a 6-in. diameter;
4. At the time of the test the age of the bond was 28 days; and
5. The data represent the average results of 81 separate tests involving:

   (*a*) Ten combinations of cleanup;
   (*b*) Four types of curing treatment;
   (*c*) Joint surfaces both wet and dry;
   (*d*) With and without mortar bedding;
   (*e*) Constant mix proportions in both lifts.

6. Data pertaining to the mixes are given in Tables 16 and 17.

Major consideration should be given to the fact that the tests represented in Fig. 26 and Table 17 were made with high-quality, low-slump concrete placed in shallow lifts with a minimum of bleeding or other segregation and formation of laitance. Fortunately, such undesirable conditions as excessive bleeding, segregation, and laitance formation in the field, which account for leakage and disintegration at the tops of lifts, just below the construction joint, in so many existing dams as noted by Mr. Steele, have been eliminated to a large extent in recent years by improved practice and control in mass concrete work.

In addition to the tests represented by Fig. 26 and Table 17, many tests have been conducted in the field at various dams by casting columns on the surfaces of concrete lifts for either pull-off or push-over tests. The results of these field tests, together with observations on the jobs, are considered essential to the proper interpretation of the laboratory results illustrated.

TABLE 16.—CONCRETE MIXES IN CONSTRUCTION JOINT BOND TESTS

Description	Concrete	Bedding mortar
Cement content[a] (barrels per cubic yard)	1.18±	2.12
Water-cement ratio (net) by weight	0.60	0.60
Average slump (inches)	2	9
Mix, by weight	1 : 3 : 5[b]	1 : 3
G/S, by weight	1.6	....
Maximum size of aggregate (inches)	1.5	....

[a] Laboratory blend, modified. [b] Approximate.

The method of curing the surfaces of concrete lifts with a membrane curing compound was included in the laboratory tests merely for purposes of comparison. As might be expected, the results clearly indicate the difficulty in completely removing the membrane compound, which is obviously necessary if complete bonding is to be obtained.

TABLE 17.—EXPLANATION OF CURING TREATMENT TESTS IN FIG. 26 (Averages, Tops Wet in All Cases[a])

Item No.	Symbols (see Fig. 26)	Air temperature (F)	Humidity	Each increment value determined by:	Mortar bedding
1		70°	50%	4 tests of 3 specimens each	With and without[a]
2		....	....	2 tests of 3 specimens each[b]	With and without
3		....	....	2 tests of 3 specimens each[c]	With and without
4		....	....	1 test of 3 specimens[d]	With mortar bed only

[a] Item 1, also with tops dry. [b] Continuous spray. [c] Clear curing compound. [d] Two-inch layer of moist sand.

Although the results shown in Fig. 26 indicate that satisfactory results (bond strength equal to 100% of the tensile strength of the concrete), under laboratory conditions, can be obtained by wire-brushed cleanup, either as an initial or final operation, the improvement in bond obtained by methods other than wire brushing is apparent. Such improvement is much more pronounced under field conditions where it is humanly impractical to secure effective and thorough cleaning of a large lift surface by wire brushing. Furthermore, the method is expensive.

Fig. 26 and Table 17 show conclusively the adequacy, under the conditions represented, of the air-water-jet method of initial cleanup, a procedure which has been popular in large dam construction during recent years. However, there are several factors on an actual job which necessitate caution and constant vigilance if comparable results under field conditions are to be secured with this method. The optimum age of the concrete at which the cutting operation should be performed varies considerably (from 3 to 12 hr),

depending upon temperature and humidity, the type of cement, and the intensity of the air-water jet employed. If performed too early or with a jet under too high a pressure, permanent disruption of aggregate particles and overcutting with unnecessary waste of concrete are probable; if too late, the removal of laitance is not always insured. Proper bonding at a deeply cut surface (which is the usual practice on the job) requires particular care and thoroughness in working the mortar layer and new concrete into the old surface, an operation which is subject to human-element variations. Under many job conditions, the surfaces of the lifts become contaminated with construction wastes and coatings of various kinds before the next lift of concrete is placed. Such conditions require sandblasting or other suitable treatment in a final cleanup operation. On a large modern dam, it is reported that contamination subsequent to the initial cleanup is prevented by frequent washing with the high-pressure air-water jet during the interval of time that the lift is exposed. Complete cost data on this method are not available, but it is known that the cost of initial cutting and subsequent jetting on this dam is more than the cost of the actual sandblasting operation, exclusive of the cost of handling the sand, used in a final cleanup method on another job. Less than 6 cu yd of sand is required for cleaning a 50-ft by 50-ft block surface.

The sandblast method of final clean-up, just prior to placing the next lift, eliminates most of the uncertainties imposed by the aforementioned construction conditions, and thus goes a long way toward providing the needed factor of safety for insuring satisfactory results. Considerable savings in construction costs have also been effected on some jobs by the use of the sandblast final cleanup as compared with an initial air-water-jet method, followed by necessary final cleaning to remove contaminating coatings.

The results shown in Fig. 26 and Table 17 indicate that exposure of the lift surfaces to laboratory air at 70°F and 50% relative humidity has little effect on bond strength as compared with moist curing. This should not be interpreted as indicating that adequate curing is not required under field exposure to hot, dry air and sun. Proper curing of lift surfaces is considered essential under job conditions.

Of particular interest are the consistently superior results obtained in the laboratory tests with the damp sand-blanket method of curing. In these tests, the sand cover was applied eight hours after placing the first lift and, therefore, was not as effective in insuring a tight bond as would have obtained had the sand been applied at two to four hours, which is indicated by supplemental tests and field experiences. The damp sand method of curing, properly applied as soon after a lift is completed as practicable, constitutes excellent insurance against many of the unfavorable conditions for satisfactory joint bonding occurring on the job by preventing the formation of laitance and calcium carbonate and by protecting the concrete surfaces from injury, contamination, and drying. In order to effect these benefits, concrete of fairly stiff consistency (−2-in. slump) must be used and the sand covering applied at a very early age of the concrete, preferably before initial set takes place. With sand curing of concrete, which is reasonably free of bleeding and segregation, and is properly placed and compacted without overworking, and with the lift surface

otherwise left in a suitable condition, about all that should be required before placing the next lift is removal of the sand and thorough cleaning with air and water jets. This method is being used on a large dam in which concrete placing has just begun. The early experiences, which may not hold under all the conditions that will be encountered throughout the life of this job, indicate that satisfactory results are being obtained at a very reasonable cost as compared with other methods. Sandblasting can always be used in conjunction with sand-blanket curing in spots where air-water washing might be inadequate.

The final step in any procedure for obtaining a good construction joint is concerned with placing the new concrete on the cleaned surface of the preceding lift. A common practice is to cover the surface with a half-inch layer of mortar in which the new concrete may be embedded. It has been demonstrated that good bond can be obtained by placing the new concrete directly on the old if the new concrete is worked very thoroughly. However, under the variable conditions and rush of construction operations, the insurance of complete bonding and contact afforded by the mortar layer is considered a very important factor and therefore its use is essential.

R. F. Blanks,[93] M. Am. Soc. C. E. (by letter).[93a]—Mr. Tyler has considered his subject of concrete control from its broadest interpretation, and has ably discussed factors in connection with the selection, production, and combination of materials and methods of construction which affect structural design conditions as well as the service performance of the finished structure.

Type of Cement.—He compares normal, modified, and low-heat cement for use in concrete dams. Normal cement is apparently included for the purpose of comparison as to physical and chemical properties. However, in the light of present-day knowledge of portland cements, the definite statement is warranted that normal cement should not be used in the construction of large masonry dams. This does not mean that reasonably satisfactory results cannot be obtained with normal (standard) cement provided construction rates are sufficiently slow and other important factors are appropriately controlled. However, modified, low-heat, or, under certain conditions, portland-puzzolan cement are so superior for masonry dams constructed with modern methods that there is no reason or justification for using standard cement.

The principal difference between standard and modified cement, if they are both ground to the same degree of fineness, is that the latter contains considerably lower percentages of tricalcium aluminate ($3CaO \cdot Al_2O_3$) and somewhat lower percentages of tricalcium silicate ($3CaO \cdot SiO_2$). Tricalcium aluminate has been shown, by extensive tests, to be an undesirable constituent of portland cement for use in hydraulic structures, and particularly so if corrosive waters are involved. Concrete made with cement containing high percentages of $3CaO \cdot Al_2O_3$ are characterized by greater volume changes, lower ultimate strengths, and reduced resistance to disintegrating agencies, than concrete made with cement containing lower quantities of this constituent. Lower $3CaO \cdot Al_2O_3$ and reduced $3CaO \cdot SiO_2$ appreciably reduce the rate of heat generated by hydration, particularly at early ages. Thus, low-heat cement in

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[93a] Received by the Secretary September 12, 1940.

combination with slow construction, or in combination with artificial cooling and rapid construction, will result in the setting heat being dissipated at a rate more nearly equal to the rate of heat generation, with attendant lower temperature differentials between interior and surface concrete, and with reduced cracking tendencies.

The use of low-heat cement without any provision for controlling concrete temperatures during construction will not necessarily eliminate cracking entirely. The flow of heat in concrete is susceptible of accurate analysis, and a decision to use low-heat in preference to modified cement should not be made until such analyses have been completed, taking into account anticipated construction conditions and practical methods of temperature control. It has been demonstrated that, with a reasonable spacing of contraction joints, large dams can be built with practically no temperature cracking. Reduction in cracking, together with elimination of joints, involves the question of economics. As improved methods of construction are devised and advances made in the development and use of materials, it is entirely conceivable that concrete dams may be built without joints and with no temperature cracking.

Materials and Concrete Production.—From Mr. Tyler's comments on concrete control, one cannot avoid being impressed by the fact that the production of concrete for a large dam under modern, rapid rates of construction is essentially a large-scale, intensive manufacturing process. Typical of such processes, the uniformity of the manufactured product, and in the case of masonry dams, the integrity of the finished structure, is dependent to a major degree upon the uniformity of the ingredient materials and the efficiency and control with which they are combined.

With the modern, efficient equipment that is available, raw aggregate materials can now be processed extensively for large jobs very economically. In large dam construction, therefore, there is little excuse for the use of aggregates which are not of satisfactory quality, grading, and uniformity, even with relatively poor raw materials. For example, uniformity of sand grading on current large jobs is easily maintained such that the variations in fineness modulus from the average do not exceed ± 0.10. Such characteristics in aggregates pay large dividends in increased cement economy and improved handling qualities of the fresh concrete, with greater economy in construction and maintenance.

Progress in the production of uniform cement has not been so satisfactory, particularly when the cement supply originates from several mills, as is usually the case for large jobs. It has not as yet been found practicable to devise cement specifications that will insure any high degree of uniformity between different brands of the same type of cement. It is not uncommon to find two brands of cement supplied under the same specifications varying as much as 100% in 28-day strength and differing materially as to heat generation, water requirement, and other mass concrete-making properties. Such conditions require extensive blending facilities on the job and close scheduling of cement shipments from the various mills in order to obtain a uniform product.

Modern batching plants have been referred to as "houses of magic." This is an appropriate designation, as the efficient manner in which up-to-date

equipment proportions the ingredient materials for mass concrete with a minimum of the human element and other variable factors, and a resulting maximum degree of uniformity, is in reality little short of magic. Developments in mixing equipment have not progressed in step with batching methods. In fact, until quite recently, equipment obtainable for mixing mass concrete containing large sizes of aggregate produced an action more nearly comparable to a ball mill than a mixer. Recently devised methods for measuring the efficiency and uniformity of mixing concrete have effected considerable improvement.

Construction Methods.—Recent advances in knowledge concerning the properties of mass concrete resulting from contemporary engineering research, as mentioned by Mr. Tyler, are gradually narrowing the gap between the designing office and the construction field.

The outstanding factor which distinguishes mass concrete from other types of concrete work is the development of cracking caused by temperature rise from the heat of hydration and subsequent cooling. To insure stress distribution and transfer in accordance with monolithic design the control of such cracking introduces problems of temperature control, foundation treatment, and limitations in construction procedure. These problems assume magnified proportions under modern, rapid-rate construction methods. It is safe to state that practically all of the unusual and varied control problems in connection with present-day mass concrete construction have been introduced as a result of speeding up the operations. For example, there were comparatively few mass-concrete problems, as engineers of today know them, involved in the construction of Crystal Spring Dam, in California, built about 50 years ago at a very slow rate. In this connection, Mr. Tyler's comments under the heading of "Concrete Control: Cracking" are particularly timely.

Construction Control.—Although little could be written concerning the details of construction control operations in a paper of this scope, Mr. Tyler's comments clearly reflect the complexity of the problems involved and the numerous items that require constant attention throughout the aggregate production, batching and mixing, transportation, placing, curing, and temperature control phases of modern concrete dam building. This, in turn, necessitates a large inspection force and adequate laboratory facilities with resulting relatively high engineering inspection and overhead costs. This mounting cost of engineering control, in terms of a percentage of the total cost, has been a source of considerable concern to engineers charged with the responsibility of building large projects. However, it should be borne in mind that many of these large projects would be economically unfeasible without the modern methods of construction. Thus, although the overhead and engineering costs are relatively high, the total costs are thereby brought within practicable economic limits, and, at the same time, a longer, more serviceable life is insured.

Concrete-Placing Buckets.—Mr. Tyler mentions some factors to be considered in connection with the use of buckets of large capacity. The large bucket is one of the many construction developments brought about by the ever-increasing demand for greater speed and, presumably, increased economy. The economy resulting from the use of large buckets is counterbalanced by either one or both of two factors: (1) Increased cement content to provide a

higher slump for a given water-cement ratio; or (2) increased cost of vibration and other consolidating operations. If the concrete from straight-side buckets of large size could be properly distributed as it is discharged rather than being dumped all in one pile, the objectionable features would be minimized. Francis T. Crowe, M. Am. Soc. C. E., has developed an 8-cu yd bucket at Shasta Dam, in California, which can be operated so that the discharge is effectively controlled and shows promise of reducing the objections to previous types of large-size buckets. Buckets of large capacity have proved to be of considerable benefit in very hot, arid locations where a rapid rate of concrete placing is necessary in order to maintain a live concrete working surface and thus avoid cold joints and unsatisfactory consolidation.

Construction-Joint Cleanup.—The writer must take issue with the statement (see heading, "Concrete Control: Cleanup, Curing, and Finishing") "that construction joints are probably the weakest locations in any concrete dam." The comments in the paper following this quotation substantiate the writer's belief that the weakest locations are in the concrete near the tops of the lifts and not in the joint itself. Many tests, conducted both in the field and laboratory using various test methods, have shown that where reasonable care is exercised in cleaning the joint surface, the bond obtained at the joint is usually stronger than the underlying concrete. This fact emphasizes the need for giving proper attention to the design of mass concrete mixes and to the placing and consolidating operations in order to avoid the undesirable conditions mentioned by Mr. Tyler. The foregoing comments indicate that a joint-treatment procedure which can be accomplished with the least cost and at the same time insure the desired results is the method that should be used. Sandblasting and washing just prior to placing the next lift accomplishes such results. Very recent experience indicates that much, if not all, of the sandblasting can be eliminated when damp sand covering is used for curing, provided that the sand is applied soon enough after finishing the lift, and that the character of the surface concrete after placing is satisfactory.

Cold-Weather Protection.—Mr. Tyler's statement (see heading, "Concrete Control: Cleanup, Curing, and Finishing") that "provision for protection against freezing for two weeks after placing may be necessary in some cases" needs some explanation. Test data as well as experience on the construction of dams in cold climates have shown that concrete is not injured by freezing after, say, three days of curing at a temperature of 50° F, provided opportunity is had at some later date for normal resumption of the hydration process. The degree of protection required depends upon the location of the concrete in the structure and upon the particular job conditions. "Hard and fast" rules applicable to all combinations of materials and conditions cannot be formulated.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

THE PRACTICE OF STATE HIGHWAY DEPARTMENTS IN THE DESIGN OF ABUTMENTS

PROGRESS REPORT OF A SPECIAL SUBCOMMITTEE OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON MASONRY AND REINFORCED CONCRETE

Discussion

BY MESSRS. E. W. WENDELL, AND STANLEY LEVITT

E. W. WENDELL,[3] M. AM. SOC. C. E. (by letter).[3a]—Although the discussion which may be advanced covering the Committee's Report would seem to be academic, it must not be lost sight of that the financial volume of work which progresses throughout this nation covering structures designed and constructed by the various states, from a span of 5 ft to a span of a few hundred feet, is a figure to be respected even in these days. There are tremendous economies which could be effected due to the application of correct design and construction by the state, not only for the work which it designs and constructs itself but also for that which is progressed by subdivisions of the state as a recognition of its methods. In too many instances the various states, including New York State, have tried to establish standards of design for bridge construction. There has been, and undoubtedly there will continue to be, some merit in the standardization of superstructure design. When this effort is introduced into substructure design, however, all the economies that engineers have endeavored to establish are immediately made nugatory. The variation in subsoil conditions, not only from state to state but from one subdivision of the state to another, is so tremendous in scope that standard designs for substructures should be eliminated from every state's activity.

The real economies before the engineer rest not in the establishment of standards but in the use of foundation knowledge. It is at this point that the great variable exists. Two bridge engineers may design superstructures that will vary but slightly in their characteristics and cost. Both of the superstructures will be entirely satisfactory. These same two engineers may vary

NOTE.—This Report was published in June, 1940, *Proceedings*. Discussion on this Report has appeared in *Proceedings*, as follows: September, 1940, by J. Wayne Courter, Assoc. M. Am. Soc. C. E.

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[3a] Received by the Secretary August 2, 1940.

tremendously in their design of the foundations for the substructure, and it is at this particular part of the work that the great variation in the cost of the bridge really occurs. An artisan dexterity covering the manipulation of standards can never be substituted for the understanding analysis of variability, with the application of creative reasoning.

Meticulous detail is advanced with respect to the design and construction of the substructures for major bridges. It is common knowledge that the loss in the so-called small bridges during any flood is far in excess of the loss in large bridges. This is rather convincing proof that more trained thought and study are invariably given to the design and construction of a major span bridge. All too often the foundations have been left to the man in charge of construction, to develop and change as he sees fit. In the height of present-day knowledge, this not only is an inefficient way to progress but positively a most uneconomical method to utilize. Much of the cost of short-span bridges is in the substructures. Changes in the original design, after construction progresses, can almost always be attributed to a lack of subsurface information or the application of erroneous subsurface information in the design.

The work which Professor MacLean has progressed cannot help but be of benefit to all of the states. To the writer's knowledge, this is the first analysis made of the foundation design for bridge structures covering the states of the Union. It is quite true that there has been considerable activity on the part of the American Association of State Highway Officials to establish specifications, and, in general, the load requirements for superstructures are recognized, in this way, by all the states. The functional relationship of the various parts of foundation design and construction, however, is not so well fixed. This is clearly indicated by the Committee's condensed report covering the activities of even contiguous states. There is such a tremendous variation in the design and construction of small bridges that it is sincerely hoped that the work which has been advanced will produce a unifying effect throughout the various states.

Over a number of years an analysis of the results obtained by subsoil exploration has convinced the writer that the organization that conducts the field activity covering sub-surface investigations should be a part of the construction organization. New York State has tried various schemes of organization in connection with its structures, from the preliminary investigation to the completed project. The method used today is satisfactory. The state is divided into ten district regions, each having a district engineer in charge of all construction in his district. All field investigations for surface and subsurface information are established by the district office. All bridge designs are made, as a centralized activity, at the headquarters office in Albany, N. Y. This places the responsibility for proper subsurface information upon the construction personnel, who must confront themselves with the situation at the time the work is placed under contract. If field and subsurface explorations are extended by separate units, it is quite difficult to obtain the desired detail information due to the endeavor to keep down costs for subsurface explorations. Nothing is more demoralizing, after the award of a contract, than for a construction organization to find that the conditions in the field differ radically from those obtained when the preliminary investigations were advanced.

There will be enough variation under the most favoarable circumstances without avoiding unified responsibility.

In addition to the method of procedure herein outlined covering the activities of the organization, New York State requires that no change be made in any foundation unless the information on the plans differs materially from the conditions found in the field. As the organization responsible for the design, the headquarters office is informed of conditions differing from the plans. This enables them to analyze the design and make such changes as can be economically established as a recognition of the alteration in the conditions disclosed in the project. This procedure is cited in detail because it avoids any lack of record covering the progression of the project, and it leaves the control of the contract in the hands of the construction organization, with complete cooperation from those responsible for the design before the change is made. The State Department of Public Works thus has a complete record, from the establishment of the preliminary field data to the completion and acceptance of the project.

For all structures of less than 5 ft in span, other than pre-cast structures, the writer is convinced that, unless rock outcrops at the site, borings should be taken with dry samples. Drive rods by themselves do not give sufficient information for a proper design, without a strong possibility of altering the design after the award of the contract. The writer prefers the blows on the casing of the wash drill, as an analysis of the density of the material indicated in the samples when dry samples are taken, to the drive rod investigation. However, if it is not possible, with the equipment that is being used, to obtain the penetration of the spoon to secure a dry sample, then drive rods should be used to avoid any misunderstanding as to the penetration of a pile in the material as located at the site of the structure. In other words, dense, cemented material will give the same results as material that is not cemented, if a wash-drill boring is established without securing dry samples. The only method that one can use to determine if this material is hard and firm is then to drive rods.

There can be no question that considerable advancement has been made during the past twenty years with regard to the knowledge of soils. There is need for more true knowledge about the bearing value of undisturbed soils. The writer does not have in mind the information of the values as a tabulation but as an understanding analysis of the soils encountered. It does not produce any advancement to know that a certain type of soil will carry a certain number of tons per square foot if the engineer fails to recognize this soil when he sees it. It is essential that the water content of the soil subsequent to the completion of the project be recognized in the design.

In the State of New York, designs have been advanced to construction with a variation in piling characteristics as a recognition of the carrying capacity of the foundation. For a great many years it has seemed to the writer that the driving of any type of displacement pile in clay to develop a frictional resistance is fundamentally wrong. However, he firmly believes that the driving of a pile in clay can, and does, materially assist the foundation in carrying the load, if the pile is driven with minimum displacement. At various localities, where the borings indicate that the clay is denser at low elevations and not immedi-

ately overlying rock, it has been found that open-ended piles, driven to an elevation predetermined by borings and then blown out and filled with concrete, create a most satisfactory medium of support. In these same localities it has been found that a displacement pile not only destroys the original value in the existing soil but creates pressures which distort the material surrounding the pile and the adjacent piles so that subsidence almost invariably occurs. Where clay overlies rock and an end bearing pile is sought and if H-sections are not used, it is essential that piles be driven open-ended to avoid distortion. Many cases of pile uplift, in which piles are driven through clay to rock, can be avoided by recognizing this feature of the design. More study can be extended to this matter but the engineers of New York State have found it to be an economical, desirable, and simple method of supporting substructures in localities where, in the past, other methods have failed. For this reason, the writer wishes to express himself, most emphatically, in favor of analyzing the characteristics of the soil when establishing the type of pile to use. He mentions this matter of driving piles into this particular character of soil—that in which the material is removed from the driven shell after the shell has been established at a predetermined elevation—because he knows there is considerable stability without a disruption of the equilibrium of the existing soil.

With the thought in mind of the utilization of piles, a pile formula should not be used for anything other than a guide. Under no circumstances should it be used as a control in driving piles. In New York State the *Engineering-News* formula is used solely as a guide. The New York State method is to consider the design, carefully, from the borings and the terrain in which the project is located. From that study an endeavor is made to arrive at a conclusion as to the type of pile to use, if piles are to be utilized. The design also recognizes if batter piles are necessary. In stream bridges, in many instances, untreated wooden piles are used. In those locations the engineer provides for a test pile to be driven before the contractor orders the piles. The form used (see Fig. 2) indicates the information which the field observer sends to the central office for a determination of the length of piles which the contractor shall order when untreated wooden piles are driven. This form indicates the make and size of the hammer, the speed of the hammer, and the number of blows per foot of driving. Always, the test piles are from 10 to 20 ft longer than the pile called for in the design. When these test-pile records are received in the Albany office they are checked against the information and data that have been acquired to reach a conclusion as to the correct length to drive in the particular foundation. As the work progresses, a daily record of all the piles driven is sent in on another type of form. This enables the central office, in all instances, to check the design against the construction contract, and it establishes a complete record, not only for future reference with regard to similar designs but as a record of the structure itself, which can be referred to should it ever become necessary or desirable to review the life history of the structure. The writer has treated this matter in rather meticulous detail because the application of this system of design and construction establishes records which are exceedingly helpful on succeeding projects and as a general guide and assistance to the entire organization throughout the state.

Unless an abutment rests on rock, considerable attention should be paid to backfilling in front of the toe before any fill is placed in back of the abutment. The writer's observations have convinced him that in many instances this is not done. If care is not exercised in placing selected material along the toe of the abutment, a forward movement is quite likely to occur when the consolidation of the backfill is progressing in layers.

For years engineers have been confronted with arguments pro and con about the so-called continuous type of open abutment. Frankly, there is considerable

Date.. Time..

Pile No.............................Location..Type of Pile.................................. (If wood, give species)

....S. H. No....................................County..

R. C. No...P. S. C. Case No...Bridge No...

Make, Type and Number of Hammer..

Measured Stroke, S. A..Strokes per Minute, D. A..

Weight of Pile or Mandrel..Weight of Gravity Hammer...

Ordered length..Length placed in leads..

0 = Elevation of bottom of footing = Elevation...

Depth of Point	Blows per Foot	Drop of Gravity Hammer	Depth of Point	Blows per Foot	Drop of Gravity Hammer	Depth of Point	Blows per Foot	Drop of Gravity Hammer
0-2			28-29			51-52		
2-4			29-30			52-53		
4-6			30-31			53-54		
26-27			49-50			72-73		
27-28			50-51			73-74		

Actual length of pile driven..

No. of blows last 6"...................................Last 3"...........................Last 2"..................................Last 1"...............................

Remarks:...

....

Sketch showing location of Pile:

FIG. 2.—FORM OF TEST PILE RECORD, NEW YORK STATE DEPARTMENT OF PUBLIC WORKS

misunderstanding about the utilization of so-called skeleton or open-type abutments. Experience covering hundreds of structures in New York State indicates that this type of design represents false economy. No one really knows the pressure on the footing of an open-type abutment, particularly subsequent to the initial movement created by subsidence, with the subsequent flattening of the rupture line. Naturally, where a structure rests on rock or excellent hardpan, or where end-bearing piles can be established to rock or hardpan, there would not be a vertical subsidence. From observations that the writer has been privileged to make, he is convinced that the load on these footings is beyond any figure which has been assumed or which may within reason be assumed. Reducing the load by increasing the size of the footings results only

in accentuating the trouble. To establish this type of design on a stream bridge, with the expectation that in extreme high water the fill will be washed out and the structure will remain and thus enable those responsible for its use to restore the fill without the loss of the bridge, might have had some basic logic thirty years ago, when a highway was a convenience. Today highways are essential lines of communication and transportation. To base a design on a rupture in the use of the highway, as a recognition of the lack of control of a stream discharge, should call for justifiable criticism from the user of the highway. In New York State it is quite essential to render 24-hr usage, 365 days a year.

STANLEY LEVITT,[4] JUN. AM. SOC. C. E. (by letter).[4a]—This Report will be found of great value to many engineers who have designed, are designing, or will design abutments in the future. The Committee has performed an invaluable function for the young engineer in presenting this Report. Here in the profound brevity of a few pages is confined the best modern practice of the composite bridge engineer in the design of highway abutments.

The new "science" of soil mechanics seems to be of much interest to bridge engineers, but they have been slow to make use of the new tools it has given the profession. Modern soil mechanics had its birth about the end of the first World War and is now completing its adolescence. Its findings, however, have not become known to the larger part of the profession, even at this late date. The First International Conference on Soil Mechanics and Foundation Engineering held at Harvard University, in Cambridge, Mass., in 1936, helped spread the results of the great researches in soils to the engineers who make their living away from the universities. The S.P.E.E. Conference on Soil Mechanics at Purdue University, in Lafayette, Ind., in September, 1940, should help in bridging the gap between research and application of soil mechanics theory. Many state bridge engineers or their representatives have attended this conference. This should go a long way in helping to clear up problem number one as suggested by many of the bridge engineers: "Develop a practical method for integrating the results of soil analysis and design." The writer would like to note his belief here that, even if this problem is successfully met, it will be no panacea for foundation ills. Because of the nature of the material with which it treats, soil mechanics will always be more a qualitative than a quantitative science. In its perfected state it will point the way to good foundations, but the engineer must remember his finest axiom: "There is no substitute for experience."

It should be recognized that it is the duty of the research side of the field of soil mechanics to point out to the busy practicing engineer where and when he can use its newly formulated earth pressure and settlement theories. Highway abutment design will react to these theories only when the designing engineer is convinced of their reliability and has been subjected to instruction in their use. This could be accomplished by means of short courses given for practicing engineers throughout the country and the publication of practical texts on the subject by leaders in the field. Simplification of theory would be

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[4a] Received by the Secretary August 3, 1940.

a requisite for these texts as practicing engineers are not prone to read very deeply into material filled with integral signs. Soil mechanics authorities can take a leaf from the column research which has finally given designers a practical and easily applicable method of column design, despite the fact that a mass of intricate mathematics was required in the development of the simple and adequate formulas now in everyday use. The prolific and valuable writing in the field of soil mechanics by many authorities to this date has been too scattered and too theoretical to be of much practical use to the bridge engineer and his staff. It must be remembered that they are busy men with a job to do.

In the past, experience was a good guide in the design of abutments and it will remain so in the future; but it must be remembered by the civil engineering profession that scientific knowledge of the action of foundations will soon be recognized by the law courts as requisite knowledge for the professional civil engineer. Failures due to earth action of one type or another were considered to be unpredictable acts of Nature in the past. Even at the present stage of knowledge of soil mechanics this attitude is untenable ethically, and it will soon be untenable legally.